इंटरनेट मानक

Disclosure to Promote the Right To Information

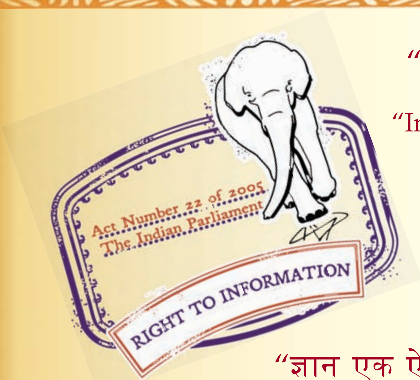
Whereas the Parliament of India has set out to provide a practical regime of right to information for citizens to secure access to information under the control of public authorities, in order to promote transparency and accountability in the working of every public authority, and whereas the attached publication of the Bureau of Indian Standards is of particular interest to the public, particularly disadvantaged communities and those engaged in the pursuit of education and knowledge, the attached public safety standard is made available to promote the timely dissemination of this information in an accurate manner to the public.

"जानने का अधिकार, जीने का अधिकार"
Mazdoor Kisan Shakti Sangathan
"The Right to Information, The Right to Live"

"पुराने को छोड नये के तरफ"
Jawaharlal Nehru
"Step Out From the Old to the New"

IS 15026 (2002): Tunneling Methods in Rock Masses -- Guidelines [CED 48: Rock Mechanics]

"ज्ञान से एक नये भारत का निर्माण"
Satyanarayan Gangaram Pitroda
"Invent a New India Using Knowledge"



"ज्ञान एक ऐसा खजाना है जो कभी चुराया नहीं जा सकता है"
Bhartṛhari—Nītiśatakam
"Knowledge is such a treasure which cannot be stolen"

BLANK PAGE

IGNORANTIA JURIS NON EXCUSAT

BRUTUM FULMEN

PROTECTED BY COPYRIGHT

IS 15026 : 2002
(Reaffirmed - 2012)

भारतीय मानक

चट्टानों में सुरंग बनाने की पद्धति – मार्गदर्शन

Indian Standard

TUNNELLING METHODS IN ROCK MASSES — GUIDELINES

ICS 93.060

© BIS 2002

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

June 2002

Price Group 10

Rock Mechanics Sectional Committee, CED 48

FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Rock Mechanics Sectional Committee had been approved by the Civil Engineering Division Council.

Tunnelling is an art practized by all engineers, geologists, planners and people. Failures should be regarded as challenges and opportunities for generating new vital knowledge and thereby increasing self-reliance in the tunnelling. The key to success is team spirit and love for rocks and nature.

The most challenging construction problem is the squeezing ground condition which is encountered in weak rock masses under high rock cover. Further the shear zones are met frequently in lower Himalayan region. Special treatment is necessary to support shear zones in the tunnels.

The design of underground excavations is, to a large extent, the design of underground support system. These can range from no support in the case of a temporary excavation in good rock to the use of fully grouted and tensioned bolts or cables with mesh; and sprayed concrete of Steel Fibre Reinforced Shotcrete (SFRS) or steel rib with concrete for the support of a large permanent civil engineering excavation.

The philosophy of design of any underground excavation should be to utilise the rock mass itself as the principal structural materials, creating as little disturbance as possible during the excavation process and adding as little as possible in the way of shotcrete or steel supports. The extent to which this design aim can be met depends upon the geological conditions which exist at site and the extent to which the designer is aware of these conditions.

There are many difficult geological conditions and extraordinary geological occurrences such as intra-thrust zones, very wide shear zones, geothermal zones of high temperature, cold/hot water springs, water charged rock masses, intrusions, etc. These are very difficult to forecast. Innovative methods of tunnelling will have to be invented. Experts must be consulted.

The composition of the technical Committee responsible for the formulation of this standard is given in Annex A.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

TUNNELLING METHODS IN ROCK MASSES — GUIDELINES

1 SCOPE

This standard provides guidelines for rock tunnelling in Himalayan and other geological regions in India. In view of the difficulties in forecasting geological formations along deep and long tunnels particularly in complex geological environment, the suggested strategy of tunnelling is such that tunnelling could be done smoothly in usually all ground conditions. This standard recommends strongly adoption of the steel fibre reinforced shotcrete to cope up with even squeezing ground conditions. The use of steel ribs should be restricted to highly squeezing or swelling rock conditions only.

2 REFERENCES

The Indian Standards given below contain provisions which through reference in this text, constitute provision of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
432	Mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement:
(Part 1) : 1982	Mild steel and medium tensile steel bars (*third revision*)
(Part 2) : 1982	Hard-drawn steel wire (*third revision*)
456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
800 : 1984	Code of practice for general construction in steel
4880 (Part 6) : 1971	Code of practice for design of tunnels conveying water: Part 6 Tunnel support
5878	Code of practice for construction of tunnels conveying water:
(Part 4) : 1971	Tunnel supports
(Part 6) : 1975	Steel lining
9012 : 1978	Recommended practice for shotcreting
9179 : 1979	Method for the preparation of rock specimen for laboratory testing
13365 (Part 2) : 1992	Guidelines for the quantitative classification systems of rock mass: Part 2 Rock mass quality for prediction of support pressure in underground openings

3 PROBE HOLES

Long tunnels, sometime pass through complex geological conditions particularly in case of deep tunnel. Geological predictions in deep tunnel are hard to make merely on the basis of surface observations. If the site conditions require, probe holes may be drilled at the face of the tunnel for about 20 m length along the tunnel alignment. These probe holes will give reliable geological and geotechnical information in advance of tunnelling. It will help in suggesting the strategy of tunnelling.

4 EFFECT OF SEISMICITY

A tunnel in a seismic area is likely to be affected near the portals and in neighbourhood of faults and thrusts. The effect is observed to be up to a distance along tunnel within ± B on both sides of the faults/thrusts, where B is span/size of the opening. The design support pressure in the affected length of tunnel may be taken as 1.25 times of ultimate support pressure [*see* IS 13365 (Part 2)].

5 TUNNEL INSTRUMENTATION

5.1 Instrumentation of tunnel openings should be done where squeezing ground condition is expected. The survival rate of tunnel instruments is generally as low as 30 percent. Therefore many sections of the tunnel should be instrumented so that enough instruments survive and reliable data are obtained. The post monitoring of support system in squeezing ground should also be carried out until support system has stabilized with time. In cases of squeezing ground conditions, observed vertical and horizontal tunnel closures should be less than four percent of tunnel width and height, respectively.

5.2 Instrumentation should also be done at other locations as per needs of the site.

6 SELECTION OF TYPE OF SUPPORT SYSTEM

6.1 Table 1 classifies various ground conditions for tunnelling. Rock bursts are rare in tunnelling for civil engineering projects. Table 2 suggests the method of excavation, type of supports and precautions for

Table 1 Classification of Ground Conditions for Tunnelling

(*Clause* 6.1)

Sl No.	Ground	Sub-Class	Rock Behaviour
i)	Competent self-supporting	—	Massive rock mass requiring no support for tunnel stability
ii)	Incompetent non-squeezing	—	Jointed rock mass requiring support for tunnel stability
iii)	Ravelling	—	Chunks or flakes of rock mass begin to drop out of the arch or walls after the rock mass is excavated
iv)	Squeezing	Mild Squeezing (u_a/a = 1-3%) Moderate Squeezing (u_a/a = 3-5%) High Squeezing (u_a/a > 5)	Rock mass squeezes platically into the tunnel and the phenomena is time dependent; rate of squeezing depends upon the degree of overstress; occurs at shallow depths in weak rock masses like shales, clay, etc. hard rock masses under high cover may experience slabbing/ poping/rock burst
v)	Swelling	—	Rock mass absorbs water, increases in volume and expands slowly into the tunnel, for example montmorill-onite clay
vi)	Running	—	Granular material becomes unstable within steep shear zones
vii)	Flowing	—	A mixture of soil like material and water flows into the tunnel. The material can flow from invert as well as from the face crown and wall and can flow for large distances completely filling the tunnel in some cases
viii)	Rock burst	—	A violent failure in hard and massive rock masses of class II type (UCS test on class II type rock shows reversal of strain after peak failure), when subjected to high overstress

where

u_a = radial tunnel closure; a = tunnel radius; and

u_a/a = normalized tunnel closure in percentage.

various ground conditions.

NOTE — Squeezing is ruled out if the observed *in-situ* maximum tangential strain (ratio of downward crown displacement to the tunnel radius) is less than the critical strain (ratio of UCS to modulus of elasticity of rock material) in case of properly supported tunnels.

6.1.1 Before taking up the design of supports, the rock load and pressure likely to act on the supports shall be estimated. The determination of rock load is complex problem. This complexity is due to inherent difficulty of predicting the primary stress conditions in the rock mass (prior to excavation) and also due to the fact that the magnitude of the secondary pressure developing after the excavation of the cavity depends on a large number of variables, such as size and shape of cavity, depth of cover, disposition of strike and dip of rock formation in relation to alignment of tunnel, method of excavation, period of time elapsing between excavation and the time when the rock is supported and the rigidity of support. These pressures may not develop immediately after excavation but may take a long period after excavation to develop due to adjustments/displacements in the rock mass.

6.1.2 In major tunnels it is recommended that as excavation proceeds, load cell measurements and diametrical change measurements are carried out so that rock loads may be correctly estimated. In rocks where the loads and deformation do not attain stable values, it is recommended that pressure measurements should be made by using flat jack or pressure cells.

6.1.3 In the absence of any data of instrumentation, rock load or support pressure may be estimated by Q system as given in IS 13365 (Part 2).

6.1.4 As the tunnels generally pass through different types of rock formations, it may be necessary to workout alternative cross-sections of the tunnel depicting other acceptable types of support systems. These types may be selected to match the various methods of attack that may have to be employed to get through the various kinds of rock formations likely to be encountered. The 'A' and 'B' lines shall be shown on these sections.

6.1.5 The support system shall be strong enough to carry the ultimate loads. For a reinforced concrete lining it is economical to consider the steel supports as an integral part of the permanent lining.

6.1.6 Temporary support system must be installed within stand-up time for safety of workmen but not too early.

7 STEEL FIBRE REINFORCED SHOTCRETE (SFRS)

7.1 Steel fibre reinforced shotcrete either alone or in

Table 2 Method of Excavation, Type of Supports and Precautions to be Adopted for Different Ground Conditions

(*Clause* 6.1)

Sl No.	Ground Classification	Excavation Method	Type of Support	Precautions
i)	Self-supporting competent	Tunnel Boring Machines (TBM) or full face drill and contra-blast	No support or spot bolting with a thin layer of shotcrete to prevent widening of joints	Look out for localised wedge/shear zone. Past experience discourages use of TBM if geological conditions change frequently
ii)	Non-squeezing competent	Full face drill and controlled blast by boomers	Flexible support; shotcrete and pre-tensioned rock bolt supports of required capacity. Steel fibre reinforced shotcrete (SFRS) may or may not be required	First layer of shotcrete should be applied after some delay but within the stand-up time to release the strain energy of rock mass
iii)	Ravelling	Heading and bench; drill and blast manually	Steel support with struts/pre-tensioned rock bolts with steel fibre reinforced shotcrete (SFRS) may or may not be required	Expect heavy loads inclu-ding side pressure
iv)	Mild squeezing	Heading and bench, drill and blast	Full column grouted rock anchors and SFRS, floor to be shotcreted to complete supporting	Install support after each blast circular shape is ideal; side pressure is expected; do not have a long heading which delays completion of support ring
v)	Moderate squeezing	Heading and bench, drill and blast	Flexible support; full column grouted highly ductile rock anchors and SFRS. Floor bolting to avoid floor heaving, to develop a reinforced rock frame. In case of steel ribs, these should be installed and embedded in shot-crete to withstand the high support pressure	Install support after each blast; increase the tunnel diameter to absorb desirable closure circular shape is ideal; side pressure is expected; instrumentation is essential
vi)	High squeezing	Heading and bench in small tunnels highly method in large tunnels; use forepoling if stand-up time is low	Very flexible support, full column grouted ductile rock anchors and SFRS. Yielding steel ribs with struts when shotcrete fails repeatedly; steel ribs should be embedded in shotcrete to withstand high support pressure; close ring by erecting invert support; floor bolting to avoid floor heaving; sometimes steel ribs with loose backfill is also used to release the strain energy in controlled manner (tunnel closure more than 4 percent shall not be permitted)	Increase the tunnel diameter to absorb desirable closure; provide invert support as early as possible to mobilise full support capacity long-term instrumentation is essential; circular shape is ideal
vii)	Swelling	Full face or heading and bench; drill and blast	Full column grouted rock anchors with SFRS shall be used around the tunnel; increase 30 percent thickness of shotcrete due to weak bond of the shotcrete with rock mass; erect invert strut. The first layer of shotcrete is sprayed immediately to prevent ingress of moisture into rock mass	Increase the tunnel diameter to absorb the expected closure; prevent exposure of swelling minerals to moisture monitor tunnel closure
viii)	Running and flowing	Multiple drift method with fore-poles; sometimes advance grouting of the ground is essential; shield tunnelling may be used in soil like condition	Full column grouted rock anchors and SFRS, concrete lining up to face, steel liner in exceptional cases with shield tunnelling	Progress is very slow. Trained crew should be deployed
ix)	Rock burst	Full face drill and blast	Fibre reinforced shotcrete with full column resin anchors immediately after excavation	Micro-seismic monitoring is essential

combination with rock bolts (specially in large openings) provides a good and fast solution for both initial and permanent rock support. Being ductile, it can absorb considerable deformation before failure.

7.2 Controlled blasting should be used preferably. The advantage of fibre reinforced shotcrete is that smaller thickness of shotcrete is needed, in comparison to that of conventional shotcrete. Fibre reinforced shotcrete is required, specially in rock conditions where support pressure is high. Use of fibre-reinforced shotcrete along with resin anchors is also recommended for controlling rock burst conditions because of high fracture toughness of shotcrete due to specially long steel fibres. This can also be used effectively in highly squeezing ground conditions. It ensures better bond with rock surface. With mesh, voids and pockets might form behind the mesh thus causing poor bond and formation of water seepage channels as indicated in Fig. 1.

7.3 The major draw-back of normal shotcrete is that it is rather weak in tensile, flexural and impact resistance strength. These mechanical properties are improved by the addition of steel fibres. Steel fibres are commonly made into various shapes to increase their bonding intimacy with the shotcrete (*see* Fig. 2). It is found that hooked ends type of steel fibres behave more favourably than other types of steel fibres in flexural strength and toughness. Accelerators play a key role to meet the requirements of early strength.

7.4 Steel fibres make up between 0.5 to 2 percent of the total volume of the mix (1.5 to 6 percent by weight). Shotcrete mixes with fibre contents greater than 2 percent are difficult to prepare and shoot.

7.5 The steel fibres are manufactured by cutting cold drawn wires as per IS 432 (Parts 1 and 2).

7.6 Some of the important parameters of steel fibres shall be:

a) Geometrical shape — as shown in Fig. 2. Length of the fibres may be 20 to 40 mm. Recommended sizes of the fibres are 25 to 35 mm × 0.40 mm ϕ
b) Aspect ratio (length/equivalent diameter) = 60 to 75.
c) Ultimate tensile strength > 1 000 MPa
d) Shear strength of SFRS = 8 to 10 MPa

7.7 Shotcrete Ingredients

7.7.1 Shotcrete ingredients in fibre reinforced shotcrete are :

a) Cement
b) Micro silica fumes (8-15 percent by mass of cement) for improving pumpability and strength and to reduce rebound

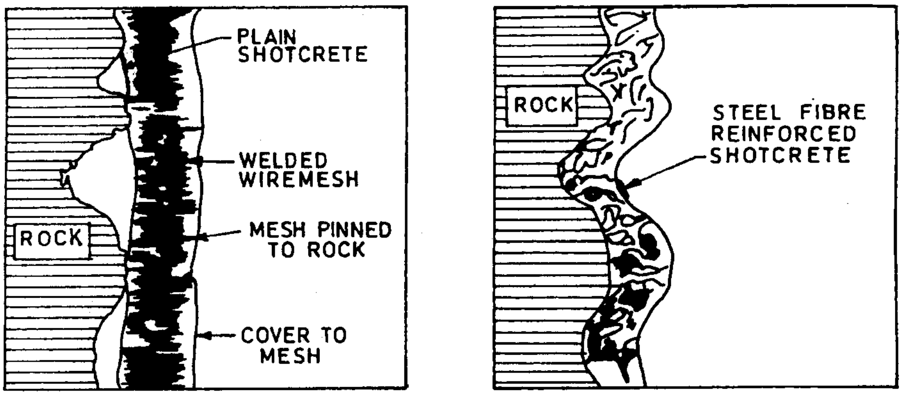
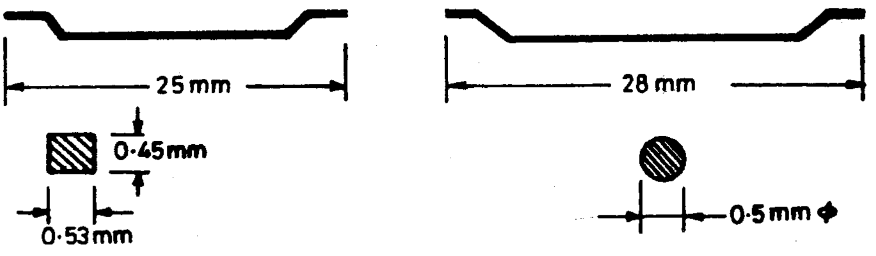


FIG. 1 DIFFERENCE IN SHOTCRETE CONSUMPTION WHEN WIRE MESH OR STEEL FIBRES



(A) TYPICAL FIBRE WITH RECTANGULAR CROSS-SECTION
(B) TYPICAL FIBRE WITH CIRCULAR CROSS-SECTION

FIG. 2 TYPICAL FIBRES USED IN SHOTCRETE WORK

c) Aggregates
d) Water
e) Hydration control agent (wet mix)
f) Super plasticizers (3-6 l/m^3) for slump increase and improvement in strength
g) Accelerators (2-5 percent by mass of cement)
h) Curing agent
j) Steel fibres

Shotcrete ingredients and properties are listed in Table 3.

Table 3 Typical Steel Fibre Reinforced Shotcrete Mix

Sl No.	Material	Mean Aggregate Size 6.35 mm Quantity kg/m³	Mean Aggregate Size 10 mm Quantity kg/m³
i)	Cement	446-558	>445
ii)	Blended sand 6.35 mm maximum size	1 483-1 679	697-880
iii)	10 mm aggregate	—	700-875
iv)	Steel fibre	39-157	39-150
v)	Accelerator	Varies	Varies
vi)	Water/cement (by weight)	0.40-0.45	0.40-0.45

7.7.2 Key to successful SFRS construction is the use of a well trained and experienced shotcrete application crew.

7.7.3 Pre-construction and post-construction testing of shotcrete shall be done for quality assurance as per IS 9012.

7.7.4 To increase the standup time, for a full front tunnel profile in poor rock quality condition (or squeezing rock conditions), spiling dowels are provided (*see* Fig. 3).

7.7.5 To stabilize the broken zone in squeezing ground conditions more than one layers of SFRS is provided (*see* Fig. 4).

7.8 Capacity of Fibre Reinforced Shotcrete

7.8.1 It is assumed that the fibre reinforced shotcrete is intimately in contact with the rock mass and having the tendency to fail by shearing.

7.8.2 Capacity of fibre reinforced shotcrete is given by

$$P_{fsc} = \frac{2 q_{fsc} \, t_{fsc}}{B F_{fsc}} \qquad ...(1.1)$$

where

q_{fsc} = shear strength of fibre reinforced shotcrete (550 t/m^2),
t_{fsc} = thickness of fibre reinforced shotcrete (m),
B = size of opening (m),
BF_{fsc} = distance between vertical planes of maximum shear stress in SFRS (m),
F_{fsc} = 0.6 ± 0.05, and
p_{fsc} = support capacity of fibre reinforced shotcrete lining (t/m^2).

7.8.3 The thickness of fibre reinforced shotcrete lining may be estimated by substituting ultimate support pressure (p_{roof}) in Equation 1.1 in place of p_{fsc}. Additional layers of shotcrete should be sprayed to arrest tunnel closure if needed.

7.8.4 Proper equipment should be used to avoid bunching of steel fibres and to ensure homogeneous mixing of fibres in the shotcrete.

7.9 Drainage System in Road/Rail Approach Tunnels Within Water-Charged Rock Mass

7.9.1 Strips of about 50 cm width along the side walls and roof should not be shotcreted to allow free seepage of ground water otherwise shotcrete is likely to crack due to building up of seepage pressure behind shotcrete in heavily water-charged formations.

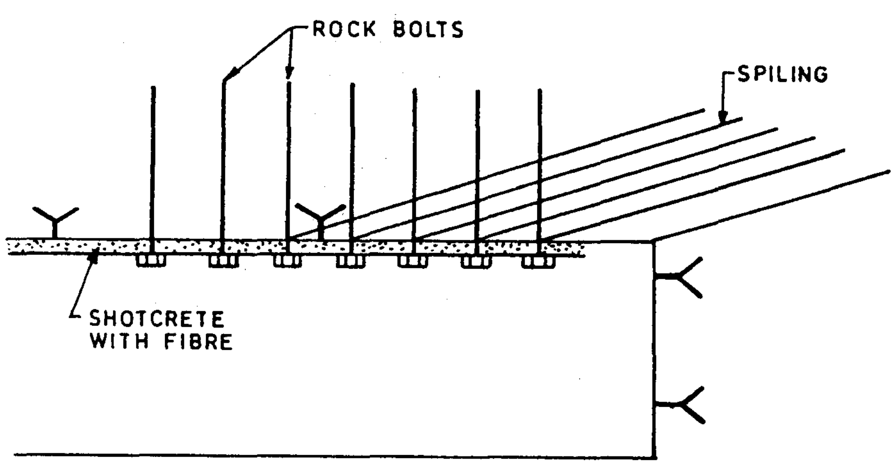


FIG. 3 ARRANGEMENT OF SPILING DOWEL WITH THE ADVANCEMENT OF TUNNEL FACE

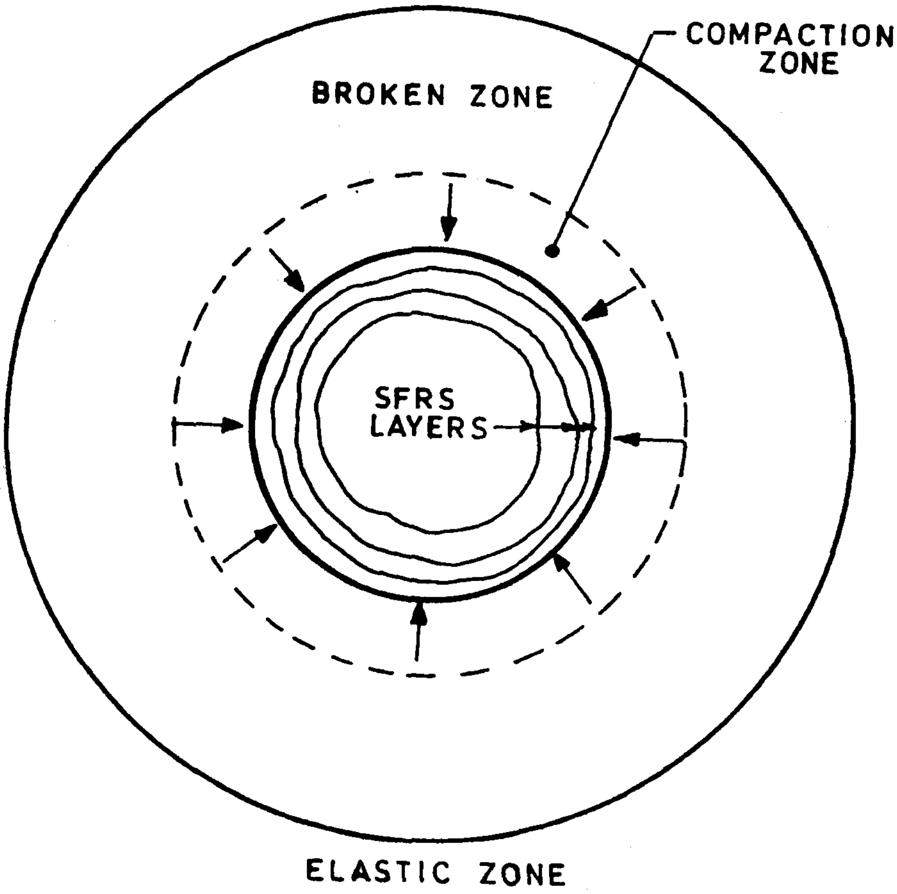


FIG. 4 STABILIZATION OF BROKEN ZONE IN SQUEEZING GROUND CONDITION

7.9.2 Drainage holes should be provided for proper drainage.

7.9.3 The catch drain should have adequate capacity to carry seepage water.

7.9.4 Swellex bolts should be used to provide support to the rock mass as it is impracticable to put grouted bolts in heavy seepage conditions.

8 TREATMENT OF SHEAR ZONE

8.1 The mean Q value may be determined, taking into consideration the breadth of weak/shear zone. The following Equation (1.2) may be employed in calculating the weighted mean Q value from the Q values for shear zone and surrounding rock mass (*see* Fig. 5)

$$\log Q_m = \frac{b \log Q_{wz} + \log Q_{sr}}{b+1} \qquad \ldots(1.2)$$

where

Q_m = mean value of rock mass quality Q for finding the support pressure [*see* IS 13365 (Part 2)],

Q_{wz} = Q value of the weak zone/shear zone,

Q_{sr} = Q value of the surrounding rock, and

b = breadth of the weak zone in metre.

Weighted mean value of joint roughness number $\bar{j}_{m}$ may be obtained after replacing log Q by appropriate value of joint roughness number in Equation (1.2). Similarly, weighted mean of joint alteration number J_{am} may be calculated.

The strike direction (θ) and thickness of weak zone (b) in relation to the tunnel axis is important for the stability of the tunnel and therefore the following correction factors have been suggested for the value of b in Equation 1.2.

8.2 Special bolting system is required for supporting the weak shear zone. Figure 5 shows a typical treatment of a thin shear zone which is thicker than 50 cm.

8.3 First the gouge is cleaned out to the desired extent. Rock bolts are then installed across the shear zone and connected with chain wiremesh. Finally, this 'dental' excavation is back-filled with shotcrete or gunite or fibre reinforced shotcrete. In wide shear zone (> 1 m) reinforcement has to be placed before shotcreting so that the reinforced shotcrete lining can withstand the heavy support pressure.

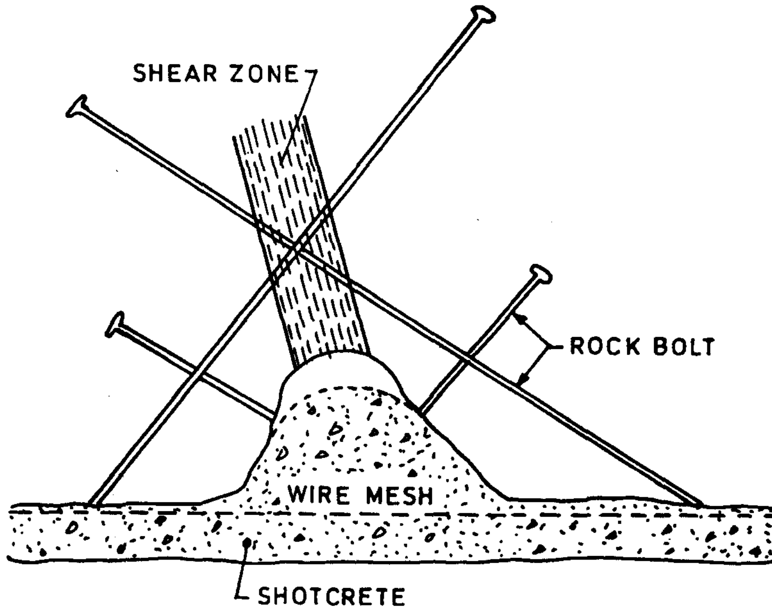


FIG. 5 TYPICAL TREATMENT OF A NARROW SHEAR ZONE

Table 4 Correction Factor for Thickness of Weak Zone (*b*)

(*Clause* 8.1)

Strike Direction (θ) to the Tunnel Axis	'*b*' to be Replaced by
90°-45°	b
45°-20°	2b
20°-10°	3b
< 10°	4b

9 GUNITING

It is recommended that guniting with cement and sand mix of 1:3 to 1:4 may be used as a form of temporary support with or without wire mesh to prevent deterio-ration of rock surface. When used in combination with rock bolts and chain link fabric, it forms a permanent support.

10 SHOTCRETE

10.1 General

10.1.1 All loose rock shall be scaled out and washed before applying shotcrete. Shotcrete for tunnel supports may be used by itself as a thin skin type reinforcement or used in combinations with rock bolts, wire mesh and other more conventional tunnel reinforcements. Details are given below:

a) All loose rocks shall be sealed out and washed before applying shotcrete.

b) Shotcrete is forced into open joints, fissures, seams and irregularities in the rock surface and in this way it serves the same binding function as mortar in a stone wall.

c) Shotcrete hinders water seepage from joints and seams in the rock and thereby prevents piping of joint filling materials and air and water deterioration of the rock.

d) Shotcrete's adhesion to the rock surface and its own shear strength provide a considerable resistance to the fall of loose rock blocks from the roof of a tunnel.

e) A thicker shotcrete layer (150 to 250 mm) provides structural support, either as a closed ring or as an arch type member.

10.2 Mix

Shotcrete shall be mixture of cement, sand and aggregate. The proportion of cement to aggregate in shotcrete may be normally 1:3 or 1:4, the aggregate being a mixture of sand and about 20 percent aggregate varying from 5 to 20 mm. The dry mixture of shotcrete shall be applied under pressure of about 3.5 kg/cm^2 by means of a nozzle. Through a separate pipe attached to this nozzle, water shall be added under pressure. A quick setting agent shall be added to the dry mixture.

10.3 Thickness

The thickness of shotcrete required depends upon the type of rock, the extent of stratification and/or joints, blockiness and also the size of the tunnel. The thickness may normally range from 50 to 150 mm and whether it should be used plain or with wire-mesh anchored to rock will depend upon the actual site conditions in each case.

10.4 Support Capacity of Shotcrete in Roof

10.4.1 It is assumed that, shotcrete is intimately in contact with the rock mass and has the tendency to fail by shearing alone. Capacity of shotcrete (p_{sc}) is given by:

$$p_{sc} = \frac{2q_{sc}\,t_{sc}}{F_{sc}B} \qquad ...(2.0)$$

where

q_{sc} = shear strength of shotcrete (300 t/m^2 in most of the cases)
t_{sc} = thickness of shotcrete (m)
B = size of opening (m)
$B\,F_{sc}$ = horizontal distance between vertical planes of maximum shear stress in shotcrete (m)
F_{sc} = 0.6 ± 0.05
p_{sc} = support capacity of shotcrete lining (t/m^2)

10.4.2 The thickness of shotcrete may be estimated by substituting ultimate support pressure (P_{roof}) for p_{sc} in Equation 2.0. Additional layers of shotcrete should be sprayed to arrest tunnel closure if needed.

11 ROCK/ROOF BOLTS

11.1 General

Roof bolts are the active type of support and improve the inherent strength of the rock mass which acts as the reinforced rock arch whereas, the conventional steel rib supports are the passive supports and supports the loosened rock mass externally. All rock bolts should be grouted very carefully in its full length. There are many types of rock bolts and anchors which may also be used on the basis of past experience and economy.

11.2 Types of Roof Bolts

11.2.1 *Wedge and Slot Bolt*

These consist of mild-steel rod, threaded at one end, the other end being split into two halves for about 125 mm length. A wedge made from 20 mm square steel and about 150 mm long shall be inserted into the slot and then the bolt with wedge driven with a hammer into the hole which will force the split end to expand and grip the rock inside the hole forming the anchorage. Thereafter, a 10 mm plate of size 200 × 200 mm shall be placed over which a tapered washer is placed and the nut tightened (*see* Fig. 6). The efficiency of the spiliting of the bolt by the wedge depends on the strata at the end of the hole being strong enough to prevent penetration by the wedge end and on the accuracy of the hole drilled for the bolt. The diameter of such bolt may be 25 mm or 30 mm. Wedge and slot bolts are not effective in soft rocks.

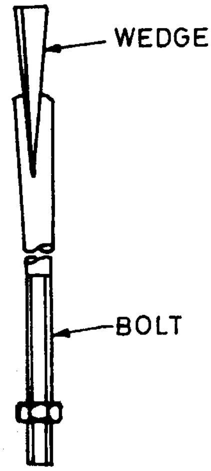


FIG. 6 WEDGE AND SLOT BOLT

11.2.2 *Wedge and Sleeve Bolt*

This consists of a 20 mm diameter rod, one end of which is cold-rolled threaded portion while other end is shaped to form a solid wedge forged integrally with the bolt and over this wedge a loose split sleeve of 33 mm external diameter is fitted (*see* Fig. 7). The anchorage is provided in this case by placing the bolt in the hole and pulling it downwards while holding the sleeve by a thrust tube. Split by the wedge head of the bolt, the sleeve expands until it grips the sides of the tube. Special hydraulic equipment is needed to pull the bolts.

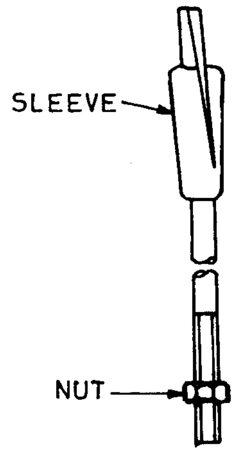


FIG. 7 WEDGE AND SLEEVE BOLT

11.2.3 *Perfo Bolts*

This method of bolting consists of inserting into a bore hole a perforated cylindrical metal tube which is previously filled with cement mortar and then pushing a plain or ribbed bolt. This forces part of the mortar to ooze out through the perforations in the tube and come into intimate contact with the sides of the bore hole

thus cementing the bolt, the tube and the rock into one homogeneous whole (*see* Fig. 8). The relation between the diameter of the bore hole and the diameter of perfo sleeve and bolts is given in Table 5.

Table 5 Diameter of Perfo Sleeve and Bolts

Dia of Bore Hole	Dia of Perfo Sleeve	Dia of Bolts
(1)	(2)	(3)
40	36	30
38	31	25
31	27	18

NOTES

1 The bolts and anchors should be checked for their straightness within ± 1 mm.

2 Pull-out tests should be done on 5 percent of bolts and anchors to check their capacity (P_{bolt}).

11.2.4 *Swellex Bolts*

These rock bolts are effective in weak rock masses charged with water.

11.3 Design

11.3.1 Immediately after a tunnel has been advanced by a length 't' (*see* Fig. 9), the rock in this section expands and settles slightly developing a double arch effect. In the longitudinal direction of the tunnel, the arch rests on the still untouched rock at the front and on the already supported portion at the back (Dashed lines in Fig. 9). The second arch effect, perpendicular to the axis of the tunnel is given by the form of the roof, which usually is an arch in tunnels. The period to which this combined arch will stand without support depends on the geological conditions, the length t and the radius of the tunnel roof, But in most cases, even in badly disintegrated rock, it will be possible to maintain this natural arch for some time, at least a couple of hours. If the natural arch is not supported immediately after mucking, it will continue to sink down slowly until it disintegrates.

11.3.2 The portion that is liable (*see* Fig. 9) to fall is generally a parabolic arch in shape having a depth $t/2$ though the loosening process will never go as deep as this if the movement is stopped by quick support. It is recommended that the bolts should not be made shorter than 't' that is twice the depth of presumed maximum loosening. The natural surrounding rock of the cavity is in this way transformed into a protective arch, the thickness of which is given by the length of the bolts 'l' which should be bigger than 't', also $l > B/4$ to $B/3$, as the *arh* also should have a certain relation to the width of tunnel. 'B' is the width of the tunnel.

11.3.3 The rock requires a prestress by bolting and the bolts should follow the static principles of prestressing in reinforced concrete as much as possible. As it is not possible to place bolts in the way of stress bars at the lower side of a beam, they should at least be given an oblique position in order to take the place of bent-up bars and stirrups (*see* Fig. 10).

11.3.4 With an arch instead of a beam, the shear forces will be greatly reduced by the vault effect but even in arch shaped roofs, shear forces may be caused by joint systems, especially by system of parallel layers like sedimentary formations, schist, etc. Hence the bolts should not only be made to exert a strong prestress to the rock but also should be set in a direction which suits best to the static demands of the geological conditions (*see* Fig. 10).

11.3.5 Just as a static member of prestressed concrete has to be prestressed before receiving the load, the rock also shall be prestressed by bolting before the load develops. This means that the space t (*see* Fig. 9) shall be bolted immediately after blasting while the next round is being drilled. The spacing between bolts/anchors should be less than half the length of bolts/anchors.

11.3.6 The pre-tension of ungrouted bolts is lost after blasting, so rock bolts shall be pretensioned again.

11.4 Capacity of Rock Bolts/Anchors

11.4.1 The capacity of reinforced rock arch is given by:

$$P_{bolt} = \frac{2 q_{crm} l}{B F_s} \qquad \ldots(3.0)$$

where

q_{crm} = minimum uniaxial compressive strength of reinforced rock mass (joint

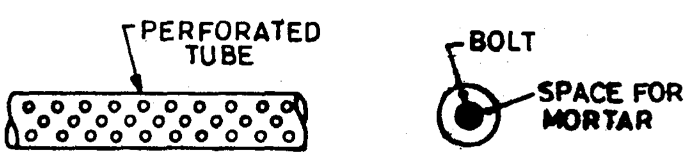


FIG. 8 PERFO BOLT

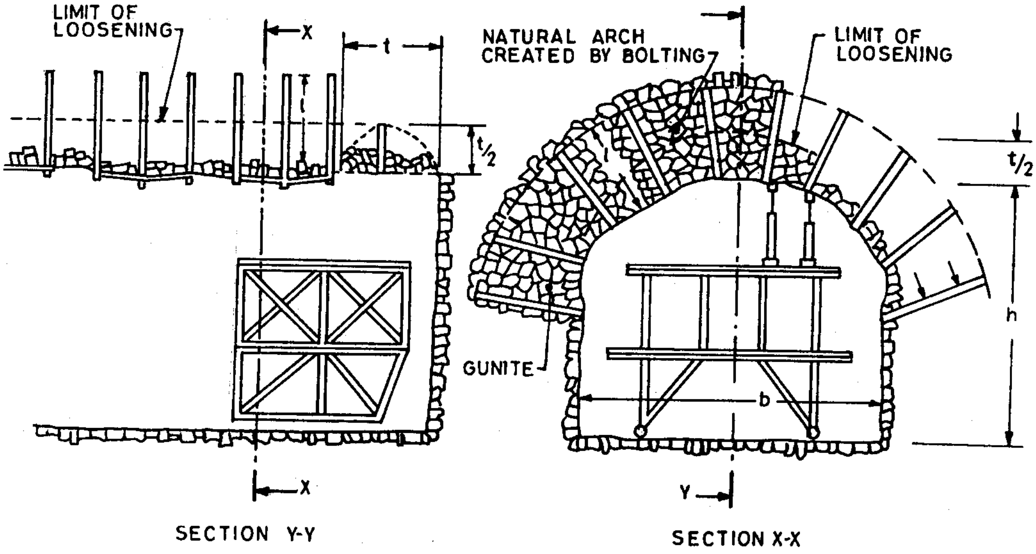


FIG. 9 DIAGRAMMATIC SECTION DEMONSTRATING OF ROOF BOLTING

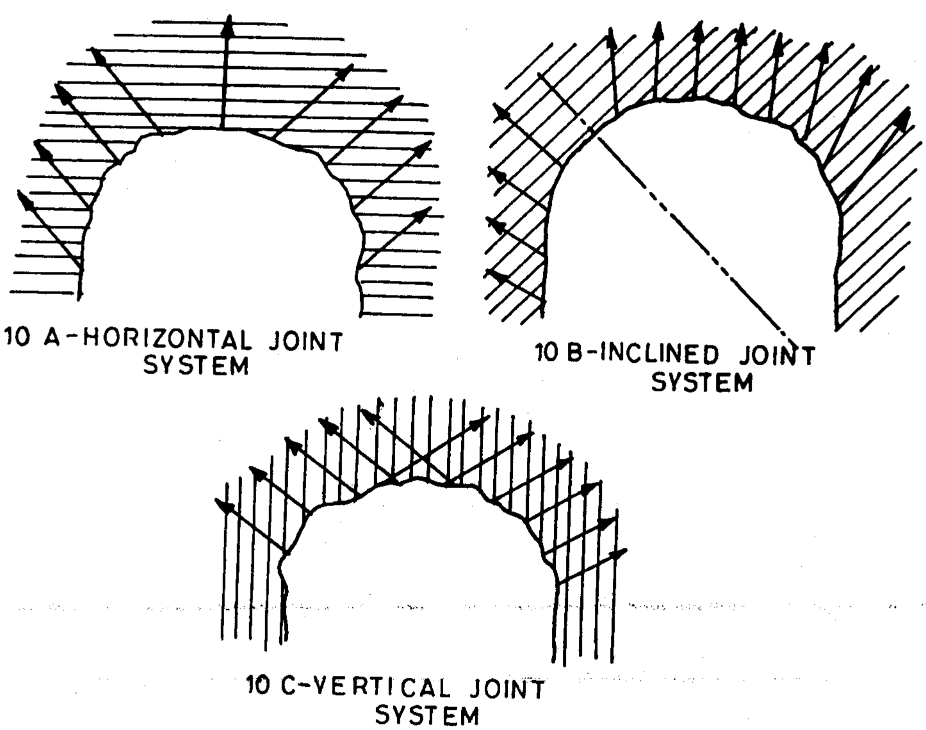


FIG. 10 ROOF BOLTING IN STRATA RUNNING AT VARIOUS ANGLE OF DIP

will be critically oriented somewhere along the arch)

$$= \left[\frac{P_{bolt}}{S_{bolt}^2} - u\right]\left[\frac{1+\sin\phi_j}{1-\sin\phi_j}\right] \geq 0 \qquad ...(3.1)$$

$\tan \phi_j$ = J_r/J_a ...(3.2)

u = seepage pressure in the rock mass (t/m^2),

P_{bolt} = tension in bolt or anchor capacity (t),

S_{bolt} = centre-to-centre spacing of bolts/anchors (m)

l' = effective thickness of reinforced arch (m)

l' = $l_{bolt} - FAL/2 - S_{bolt}/2 + S_{rock}$ ...(3.3)

l' = $l_{bolt} - FAL/2 - S_{bolt}/4 + S_{rock}$ (in case of mesh reinforced shotcrete) ...(3.4)

l' $\geq$ l_{bolt} – FAL/2 ...(3.5)

FAL = fixed anchor length (m) of anchors to develop pull out capacity of P_{bolt}

= 100 × diameter of anchor bars

= < l m in case of mechanically anchored bolts

S_{rock} = average spacing of fractures in rock (m)

l_{bolt} = length of bolt or anchor (m)

B = size of opening (m)

F_s = mobilization factor of bolt/anchor

F_s = 3.25 p_{roof} 0.10 (for mechanically anchored and pre-tensioned bolts) ...(3.6)

F_s = 9.5 p_{roof} – 0.35 (for full column grouted anchors) ...(3.7)

p_{roof} = ultimate support pressure in roof of tunnel (t/m²).

J_r = joint roughness number

J_a = joint alteration number

The spacing and length of bolts should be so chosen that the estimated capacity of rock bolts/anchors (P_{bolt}) is equal to ultimate support pressure (P_{roof}).

11.4.2 Full-column grouted bolts are more efficient in poor rock conditions according to Equation 3.7 than ungrouted pre-tensioned bolts. For permanent supports, all bolts should be grouted.

12 STEEL RIBS

12.1 Rock tunnel support systems of steel may be generally classified into the following principal types:

a) Continuous ribs (*see* Fig. 11A),
b) Rib and post (*see* Fig. 11B),
c) Rib and wall plate (*see* Fig. 11C),
d) Rib, wall plate and post (*see* Fig. 11D),
e) Full circle rib (*see* Fig. 11E), and
f) Yielding arch steel rib with socketed joints.

NOTE — Invert strut may be used in addition, with types (a) to (d) where mild side pressures are encountered (Fig. 11F) or squeezing ground is met.

12.2 Selection of Type of System

12.2.1 *General*

While choosing the type of support system, the following factors shall be considered :

a) Method of attack,
b) Rock characteristics, its behaviour and development of rock load, and
c) Size and shape of the tunnel cross-section.

12.3 Selection of Supports with Reference to Surrounding Strata and Shape of Tunnel

12.3.1 *Continuous Ribs*

This type can be erected more rapidly than the other types and is generally recommended for use in rocks whose bridge action period is long enough to permit removal of gases and mucking. Invert strut may be used in addition where mild side pressures are encountered and squeezing ground is met.

12.3.2 *Rib and Post*

This type is generally recommended for use in tunnels whose roof joins the side walls at an angle instead of a smooth curve. It may also be used in large tunnels, such as double-track rail road or two-lane highway tunnels, to keep the size of the rib segments within handling and transporting limitations. Invert strut may be used in addition where mild side pressures are encountered and squeezing ground is met.

12.3.3 *Rib and Wall Plate*

This type is generally recommended for use in tunnels with a large cross section with high straight sides through good rock or in large circular tunnels, where it is possible to support the wall plate by pins and where the strata below the wall plate does not require support. This type of support may also be used for tunnelling through spalling rock, provided spalling occurs only in the roof. However, in many cases it is extremely difficult to establish adequate support for the wall plate at any point above the floor-line due to irregularity of the overbreak.

12.3.4 *Rib, Wall Plate and Post*

This type of support permits post spacing to be different from the rib spacing and is generally recommended for use in tunnels with high vertical sides. Invert strut may be used in addition, where mild side-pressures are encountered and squeezing ground is met.

12.3.5 *Full Circle Rib*

This type is recommended for use in tunnels in squeezing, swelling and crushed, or any rock that imposes considerable side pressure.

12.4 Spacing of Ribs

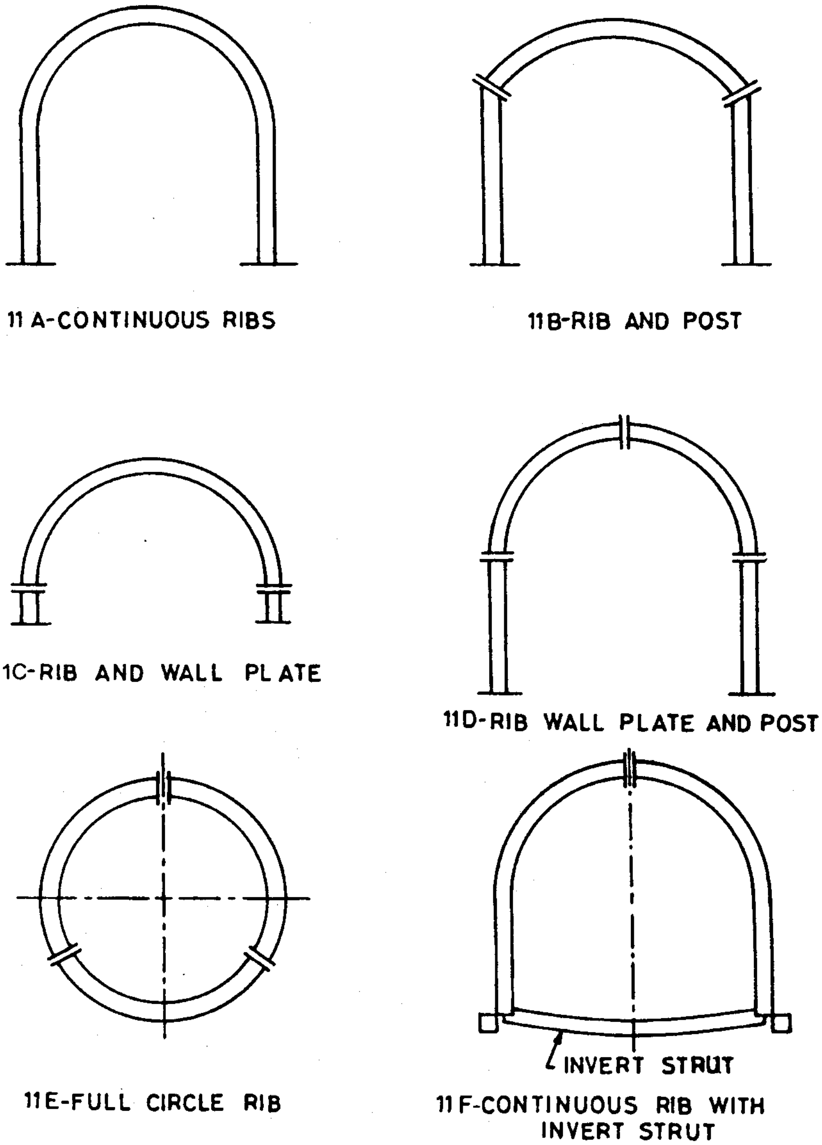
Spacing of ribs is given by :

$$S_{rib} = \frac{p_{rib}}{Bp_{roof}} \qquad ...(4.0)$$

where

p_{rib} = steel rib capacity (t),

S_{rib} = spacing of rib (m),



NOTE — Joints have not been shown in Fig. 11 (A) to 11 (F) and may be located depending upon the construction and fabrication convenience.

FIG. 11 TYPES OF STEEL SUPPORT SYSTEMS

B = size of opening (m),

p_{roof} = ultimate support pressure in roof of tunnel (t/m^2).

NOTE — The support capacity of steel ribs (p_{rib}) should be so chosen that the minimum clear spacing between ribs in poor rock condition is 10 cm more.

12.5 Selection of Steel Ribs Supports with Reference to Method of Attack for Tunnel

12.5.1 All the types of supports mentioned in **10.3** are suitable for the full face method of attack for rock where required bridge action period for providing supports is available. Detailed guidelines are given in Table 2.

12.5.2 Rib and wall plate or rib wall plate and post are suitable for heading and bench method. The rib, wall plate and post type may be supplemented by truss panels or crown bars, which are accessories developed to handle heavy loads that come quickly by supporting the intervening ribs while the bench is shot out.

12.5.3 Where it becomes necessary to drive first the top heading only due to bad roof conditions the rib and wall plate type of support is generally recommended for use in the heading, and post may or may not be used when the bench is taken out depending on rock conditions. If the post is used the excavated shape will be '*D*' shaped while without the post the excavated shape will be a mushroom.

12.5.4 Where the side drift method is used for driving a large size tunnel in poor rock conditions, the rib, wall plate and post type of supports are recommended;

the wall plate, however, being flat. The posts and wall plates are erected in the drift which is driven ahead at each side at sub-grade (*see* Fig. 12).

12.5.5 Where extreme poor rock conditions are encountered, breakups to the crown may be made, leaving a central core. Temporary posts may be quickly placed between the core and the roof at dangerous spots, and crown bars may be slid forward to quickly catch up the roof. The roof ribs should then be placed on the wall plates and securely blocked to take the roof load, after which the temporary posts may be removed.

12.5.6 The side drifts themselves usually need support which should be removed just prior to shooting out the core of the main tunnel and re-used ahead. The support system used for the drifts is hybrid. The outer side consists of the posts and wall plates which later becomes a part of the support for the main tunnel, whereas the inner side is a continuous rib (*see* Fig. 12).

12.6 Type of Support for Shafts

For shafts, usually the full circle rib or segmental ribs are recommended depending upon the slope and rock conditions. In vertical shafts, ribs may be hung from top by hanger rods and blocked and packed. The spacing of hanger rods may be worked out as in the case of tie rods keeping in view that they shall be strong enough to support the weight of ribs.

12.7 Components of Tunnel Supports

Design of various components of tunnel supports shall be done in accordance with IS 4880 (Part 6).

12.7.1 *Ribs*

Ribs may be made of structural beams. H-beams or wide flange beams should be preferred to I-beams, as the wider flanges provide more surface for blocking and lagging, and the section has greater resistance against twisting. Channel sections are not recommended as their asymmetrical section is prone to twisting, and their flanges are narrow. In small tunnels, however, channel bent about their minor axis may be used under ordinary loads. When choosing the steel section with different weights for arch rule of posts, it is advisable to select beams of equal depth.

12.7.2 *Posts*

The spacing between the posts may be normally equal to that of the ribs. However, by inserting wall plate between the ribs and the posts the spacing of the posts can be made independent of ribs. The posts should be made of H-section. The depth of these should normally be the same as that of the ribs, though in many cases they may be of lighter sections as long as no side pressure is present.

12.7.3 *Invert Struts*

Where the side pressures are present and tunnel section has not been converted to a full circle, it is necessary to prevent the inward movement of the rib or posts feet and in such cases, invert struts should be provided at tunnel subgrade. They should be so attached to the vertical members that they receive the horizontal pressure. They may be curved to form an inverted arch where there is upthrust from the floor.

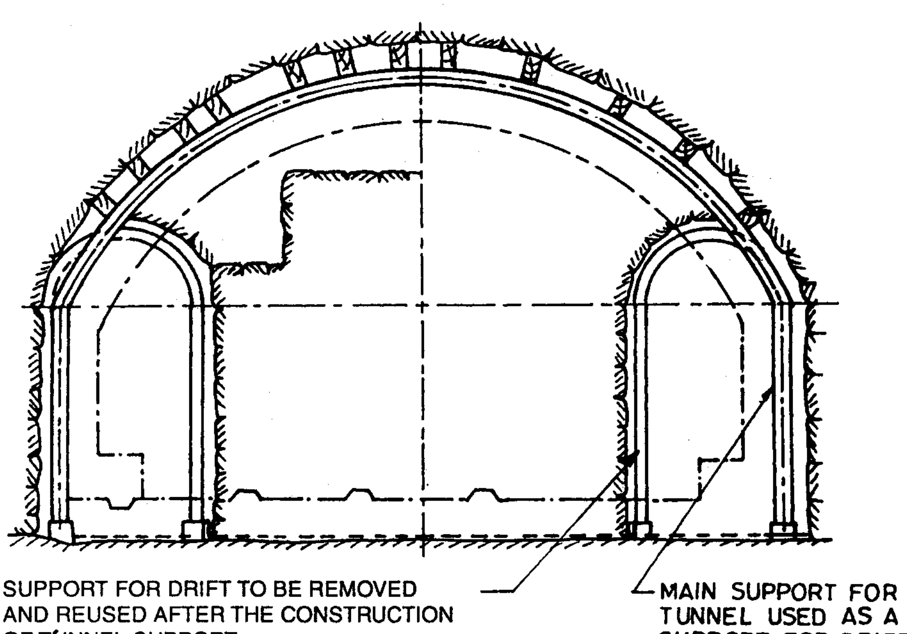


FIG. 12 SIDE DRIFT METHOD

12.7.4 *Wall Plates*

12.7.4.1 The following types of wall plates are commonly used:

a) Double beam,
b) Single beam, and
c) Flat.

12.7.4.2 The double and single beam wall plates which are intended to resist bending in vertical planes are recommended for use to transmit the loads from the ribs on to block or posts with a spacing different from that of the ribs. Flat wall plates merely serve as an erection expedient and a convenient surface for horizontal blocking, their resistance to bending in vertical planes being very small, whenever flat wall plates are used, a post shall be placed under each rib.

12.7.4.3 Double beam wall plates may be made of two I-beams placed side by side, webs vertical with about a 100 mm space between flanges to give access to the clamping bolt and admit concrete (*see* Fig. 13). The beams should be spaced by vertical diaphragms welded under each rib seat. Ribs and posts should be clamped by toggle plates and bolts, thus avoiding the time required for matching bolt holes. This method of attachment also permits variable spacing of either or both the ribs and the posts. This type of beam provides a broad surface of contact for blocking and to engage ribs and posts. Its box section makes it stable with respect to rolling and twisting.

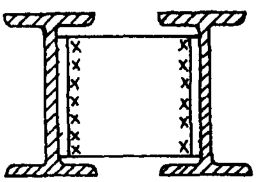


FIG. 13 DOUBLE BEAM WALL PLATE

12.7.4.4 Single beam wall plates may be H-beams, with web vertical. To enable them to transmit vertical loads from rib to post, they may be reinforced at each rib seat with vertical T-shaped plates, if necessary (*see* Fig. 14). Attachment of ribs and posts shall be made by bolting through the flanges.

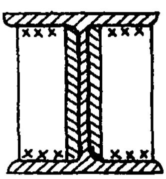


FIG. 14 SINGLE BEAM WALL PLATE

12.7.4.5 Flat wall plates may be I-beams or wide flange beams used with their webs horizontal. They function merely as a cap for the posts and a sill for erecting roof ribs. The web shall be punched with vent holes which also helps to pass reinforcing rods if the concrete is reinforced.

12.7.5 *Crown Bars*

12.7.5.1 Crown bars may be built up of double channels (*see* Fig. 15) or may be H-beams or square timber beams. They are located parallel to the axis of the tunnel either resting on the outer flanges of the ribs already erected (*see* Fig. 16A) or attached to the ribs in hangers (*see* Fig. 16B). Crown bars are an accessory, a construction expedient intended to carry loads till the rib sets are erected and the loads permanently transferred to them. They have one of the two functions to perform to support the roof immediately after ventilation and thereby gain time for the installation of ribs and to support the roof or roof ribs over the bench shot thereby relieving or supplementing the wall plates.

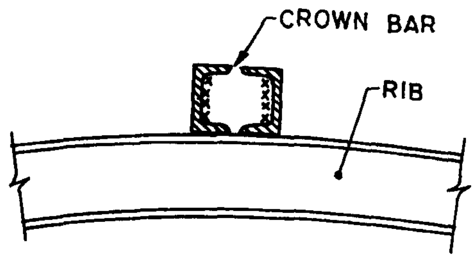


FIG. 15 CROWN BAR

12.7.6 *Truss Panels*

12.7.6.1 These are accessories for use with the combination of rib and post types of support, for the heading and bench or top-heading methods of attack and heavy roof loads. Their purpose is to form, in combination with the ribs, a truss to span the gap produced by the bench shot.

12.7.6.2 The truss panels should be attached to the inside face of the ribs for a distance of one or more ribs ahead of the bench shot (*see* Fig. 17) and should be left there until posts are installed, at that time they should be removed and sent up ahead. Attachment should be by means of only two bolts at each rib. The truss thus formed may even be designed to carry the roof over two bench shots making it more convenient to get in the post.

12.7.6.3 When truss panels are used, no wall plates are required although the flat wall plate may be used to keep the lower ends of the ribs lined up laterally if it is difficult to block the individual ribs against the rock. The truss panels eliminate the need for wall plate for drifts.

12.7.7 *Bracing*

12.7.7.1 Longitudinal bracing or struts increase the resistance of ribs and posts to buckling about their

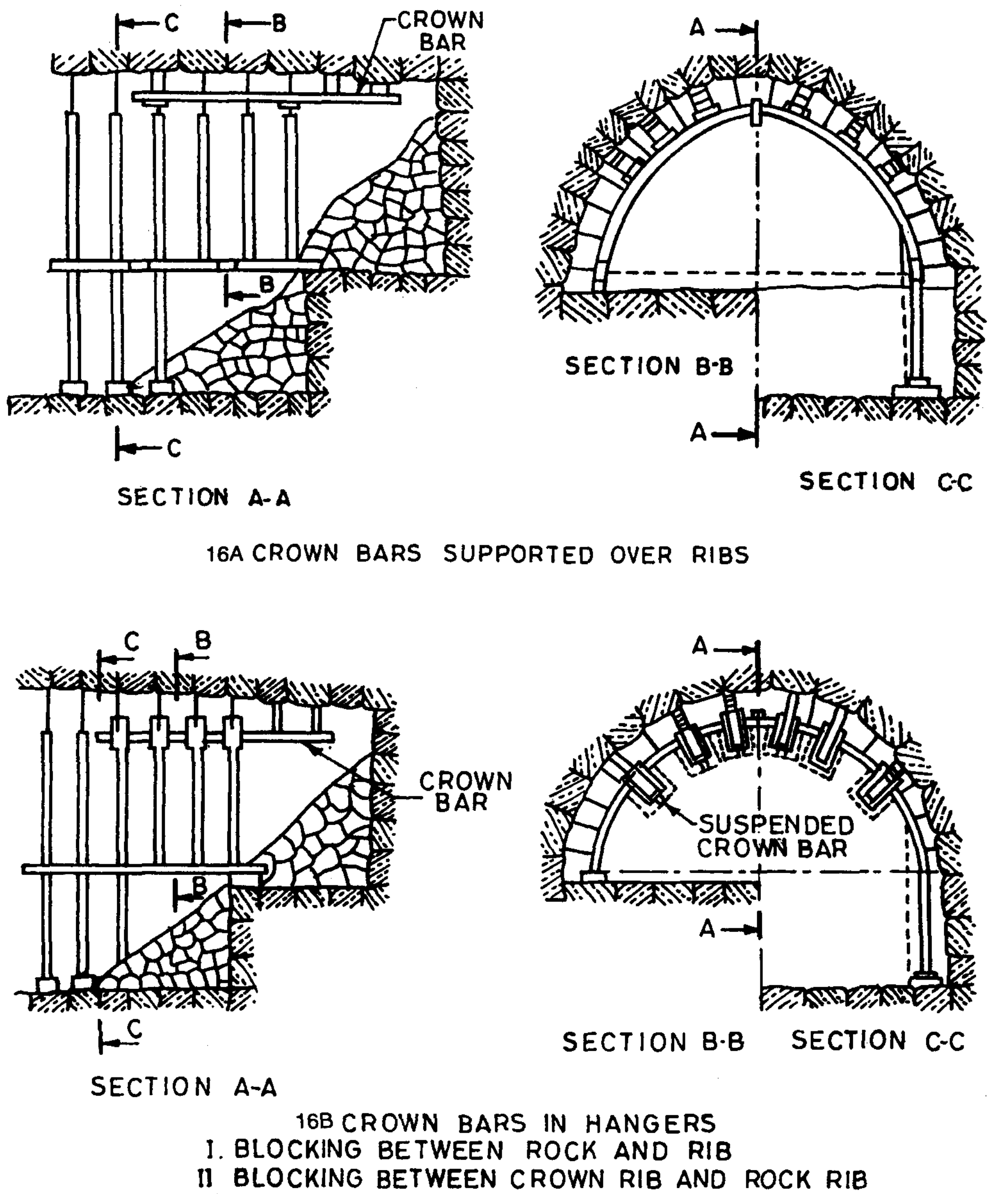


FIG. 16 METHOD OF SUPPORT FOR CROWN BARS

minor axis and prevent a displacement of these set members during blasting. If the space between the ribs or post is bridged by lagging which is firmly attached to the webs, no such bracing is required. The most common types of bracing are the rods and collar braces (*see* Fig. 18). The braces may, however, be placed as convenient between the rods.

12.7.7.2 Tie rods may be 15 mm to 20 mm rods, with thread and two nuts on each end. The length shall be at least 100 mm more than the spacing of the ribs. The spacing of the rods shall be kept such that slenderness ratio l/r for ribs is not greater than 60, where l is the spacing of the rods, and r is the least radius of gyration of the ribs. Collar braces may be usually pieces of timber, 75 mm × 100 mm, 100 mm × 150 mm, 150 mm × 150 mm or any conventional size. Holes in pairs shall be provided in the web of ribs and posts for the tie rods. Collar braces shall be set in the line between ribs, tie rods inserted and the nuts tightened. Wooden collar braces should be removed before placing final lining.

12.7.7.3 Spreaders (*see* Fig. 19) which are additional braces may be angles, channels, or I-beams with a clip angle or plate either bolted or welded on each end to the ribs. These are left in the concrete. In tunnels having steep slopes tie rods may be replaced by spreaders.

12.7.8 *Blocking*

It is generally done by using timber pieces tightly

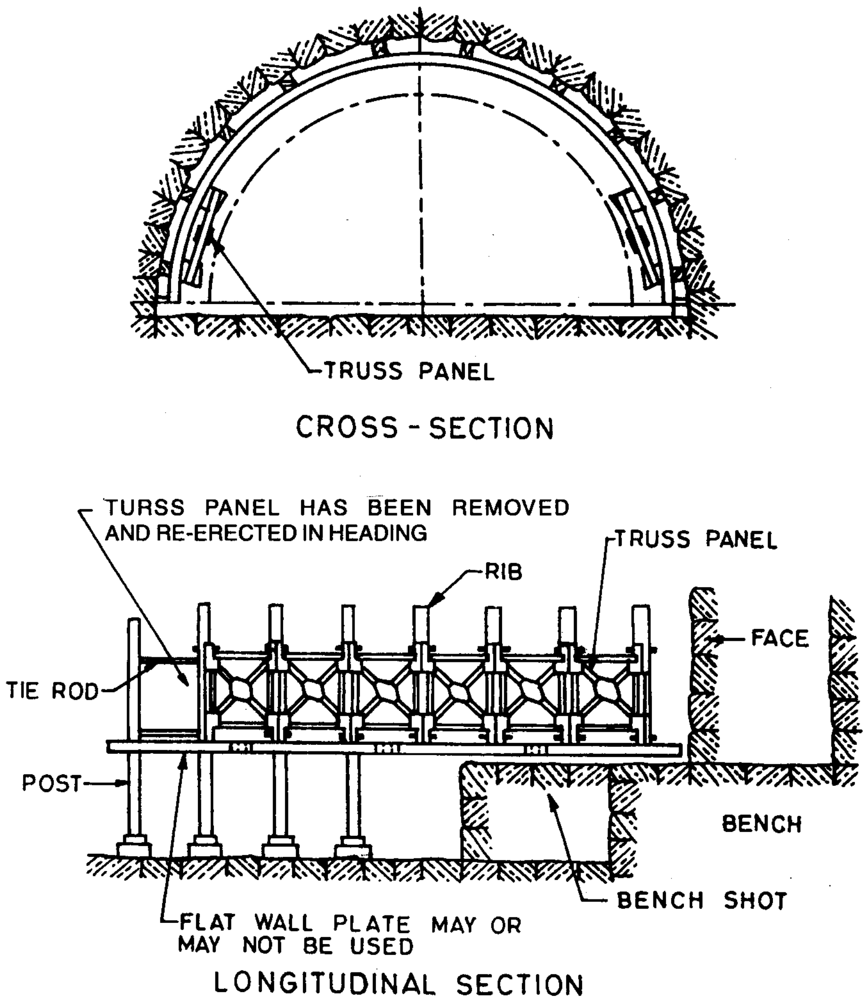


FIG. 17 TRUSS PANEL

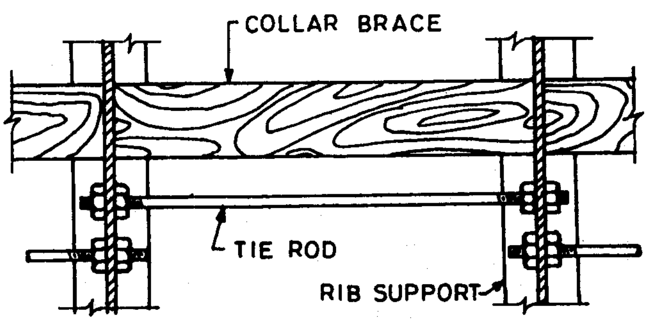


FIG. 18 COLLAR BRACE AND TIE RODS

wedged between the rock and the rib. These timber pieces are to be removed prior to concreting.

12.7.9 *Lagging*

12.7.9.1 It performs one or more of the following functions :

a) To provide protection from falling rock or spalls;
b) To receive and transfer loads to the rib sets;
c) To provide a convenient surface against which to block in case it is not convenient to block directly against the rib, because of irregular overbreak;
d) To provide a surface against which to place back packing;
e) To serve as an outside form for concrete lining, if concrete is not to be poured against the rocks, and

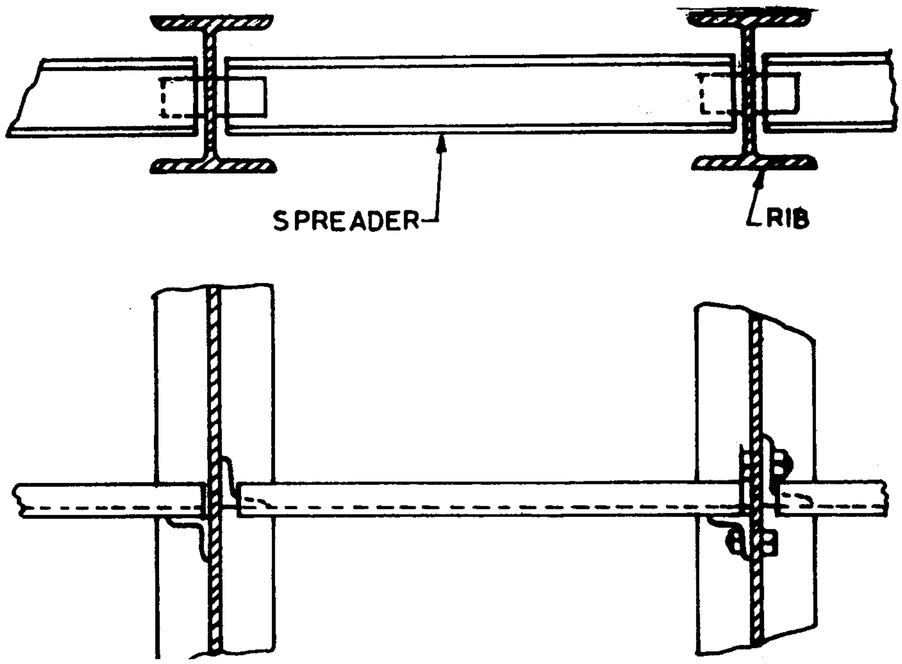


FIG. 19 SPREADER DETAILS

f) To divert water, and to prevent leaching and honey-combing of concrete.

12.7.9.2 Lagging may be either of steel, precast concrete or timber. Steel laggings may be made out of channels, beams, beams and plates and liner plates (*see* Fig. 20 and 21). Liner plates, which are pressed steel panels may also be used with or without ribs depending upon the rock conditions. It is recommended that use of timber in underground work should be minimized as far as practicable, since timber once fixed can be rarely removed safely and likely to deteriorate and prove to be a source of weakness. Total prohibition of timber is, however, not practicable.

12.7.9.3 The spacing of lags shall be closest at the crown, increasing down to spring line. On the side only an occasional lag should be used, if necessary. Close lagging should be employed where rock conditions make it necessary.

12.7.10 *Packing*

The function and type of packing depends on the rock condition. In dry tunnels through jointed rock, packing is only used to fill large cavities produced by excessive overbreak. In broken, crushed or decomposed rock it serves to transfer the rock load to the lags, thereby acting as a substitute for excessive blocking. In squeezing rock it provides continuous contact through the laggings with the rib sets. In jointed water-bearing rock it has primarily the function of a drain.

12.7.11 *Dry Packing*

Dry pack, which usually consists of tunnel spoil (hard) shoveled or hand packed into the space between the lagging and the rock, is recommended for use only where excessive rock loads are not likely to develop.

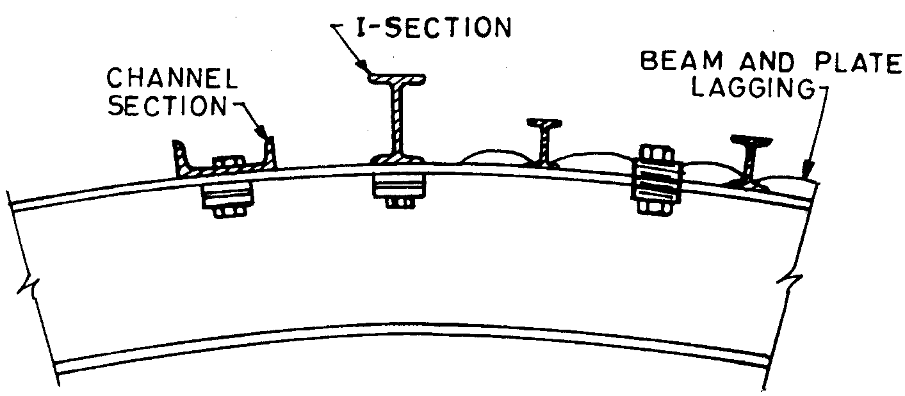


FIG. 20 TYPES OF LAGGING

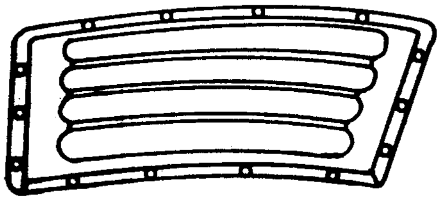


FIG. 21 LINER PLATE

It should be placed simultaneously with the erection of the lagging. Starting at the lowest point, a few lags should be placed and tunnel spoil (hard) shoveled in behind. This procedure should be carried up to the crown at which point it is necessary to pack endwise.

12.7.12 *Concrete Packing*

It is recommended for use where considerable rock loads are anticipated. However, its use is not recommended in case the tunnel supports are designed as yielding supports. Concrete packing may be M10 concrete conforming to IS 456. It may be placed by manual labour or by pneumatic placer to the extent possible. Where excessive loads are anticipated, concrete packing should start from the inner flanges of the steel support so as to embed the whole steel supports in concrete. In such cases it is recommended that precast concrete may be used as additional lagging between two adjacent ribs so as to serve the purpose of form work.

12.8 Factors Determining Spacing and Layout of Supports

12.8.1 The strength and spacing of rib system shall be determined by rock load. For a given rock load and cross-section of tunnel the spacing between the ribs shall be worked out. The spacing of the ribs should be so chosen that the sum of the cost of ribs and lagging is minimum. For preliminary designs in ordinary rocks the depth or rib section may be taken as 60 to 75 mm for every 3 m of bore diameter with ribs spaced at about 1.2 m for moderate loads, 0.6 to 1.0 m for heavy loads and 1.6 m for very light loads. Whether the ribs shall be of two or more pieces will depend on the lengths of the structural steel members available.

12.8.2 For junctions, plugs and control chamber, etc, supports shall be designed to suit special features of the work and its construction procedures.

12.8.3 Concrete blocks of suitable size and thickness may be provided, if necessary, below the vertical lags to provide adequate bearing area to the rib.

12.8.4 In tunnels, where supports are not to used as reinforcement, they may be installed plumb or perpendicular to the axis of the tunnel depending on tunnel slope and as found convenient. However, where supports are to be used as reinforcement in pressure tunnels, they may be installed at right angles to the tunnel axis, if practicable.

12.8.5 For speed of erection of supports it is essential to :

a) design support system with a minimum number of individual members, consistent with construction convenience;
b) design the joints with utmost simplicity and with minimum number of bolts; and
c) fabricate the members with sample bolt and wrench clearances. Time consuming close fits shall be avoided.

12.9 Design

12.9.1 *General*

The deign of steel components for tunnel supports shall generally conform to IS 800.

12.9.2 *Stresses*

Permissible stress in steel shall be in accordance with IS 800.

12.9.3 *Ribs*

Rock load may be assumed to be transmitted to the ribs at blocking points, each blocking point carrying the load of the mass of rock bounded by four planes, namely, the longitudinal planes passing through mid-points between the blocks and transverse planes passing through mid-points between the ribs to a height equal to the acting rock load. The blocking points may be assumed to be held in equilibrium by forces acting on it in the same manner as panel points in a truss. Values of thrust in the rib may be computed by drawing the force polygon. Ribs shall be designed for the thrust thus computed taking into account the eccentricity of this thrust with reference to the rise of the arc between the blocking points which will cause flexural stresses in addition to direct stresses.

12.9.4 *Tie Rods*

[*See* IS 5878 (Part 4)]

12.9.5 *Lagging*

Lagging may be designed for the load of rock mass as shown in Fig. 22 [*see also* IS 5878 (Part 4)].

12.9.6 *Liner Plates*

Where only liner plates are used for support their cross-sectional area and their joints shall be designed to transmit the thrust (*see* **12.9.3**). It shall be ensured that liner plates are thoroughly in contact with the ribs so that passive resistance is developed and no bending moments are induced. For tunnels with more than 3 m diameter liner plates may be reinforced by

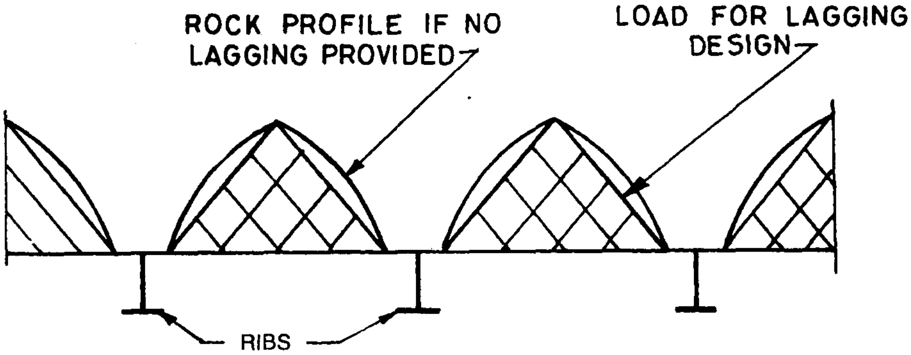


FIG. 22 LOADING DIAGRAM FOR LAGGING

I-beams. Where liner plates do not form a ring and are used in top half ribs they shall be designed as lagging [*see also* IS 5878 (Part 4)]. The thickness of liner plates may vary from 5 to 10 mm depending upon the size of bore and loads encountered. The diameter of bolts may vary from 12 to 15 mm.

12.9.7 *Joints*

Butt joints should be preferred to spliced joints. In soft grounds and poor rock, welding of joints in the field should be avoided as far as possible.

13 GROUTING

13.1 General

Grouting is carried out to fill discontinuities in the rock by a suitable material so as to improve the stability of the tunnel roof or to reduce its permeability or to improve the properties of the rock. Grouting is also necessary to ensure proper contact of rock face of the roof with the lining. In such cases the grouting may be done directly between the two surfaces or the process of grouting may be used to fill the voids in the rubble packing where used. All the three types of grouting may not be required in all cases. The grouting procedures should aim at satisfying the design requirements economically and in conformity with the construction schedules. The basic design requirement generally involve the following:

a) Filling the voids, cavities, between the concrete lining and rock and/or between the concrete and steel liner;
b) Strengthening the rocks around the bore by filling up the joints in the rock system;
c) Strengthening the rock shattered around the bore;
d) Strengthening the rock, prior to excavation by filling the joints with cementing material and thus improving its stability; and
e) Closing water bearing passages to prevent the flow of water into the tunnel and/or to concentrate the area of seepage into a channel from where it can be easily drained out.

13.2 Before drawing up the specifications for grouting the design requirements shall be established. In general for all underground structures, grouting is an universal requirement for all concrete lined tunnels. Design requirements are only to establish the maximum allowable pressure at which this grouting is to be carried out and the zone in the cross-section and the spacing of grout holes, both in the direction of the tunnel. For consolidation grouting the design requirement to be established is the thickness of the rock stratum around the bore that is to be strengthened and made impermeable, the pressure and the spacing pattern of holes. This will determine the depth to be grouted.

13.3 For tunnels, the commonly used procedures are to continue grouting to refusal at the design pressure in each hole or to interrupt the grouting if there is heavy intake with little or no pressure build up, indicating very open structures and escape of grout over a long distance.

13.3.1 *Pattern, Depth and Arrangement of Holes*

13.3.1.1 *Backfill or contact grouting*

13.3.1.1.1 The purpose of backfill grouting is to fill the space left unfilled with concrete between the concrete lining and the rock surface in the arch portion of any tunnel or cavity due to shrinkage of concrete.

13.3.1.1.2 Backfill grouting should be done after the concrete in lining has gained strength. The period of waiting may be from 21 to 28 days. In case of precast lining segments this restriction of waiting will not apply and the grouting may be done immediately after the segments are erected.

13.3.1.1.3 Backfill grouting is limited to the arch portion of a tunnel or cavity and is not required in case shaft of the concrete is poured in a concrete ring.

13.3.1.1.4 The grout holes at the crown should be

placed 5° to 10° from the crown, alternately to the left and right of the crown. In addition to the crown hole there shall be two more holes, one on either side of the crown. These holes will be 90° apart and will be located such that one of these two holes is at 22.5° from the crown alternately on the right and left of the crown. Such sections shall be, normally, 3 m apart. The exact location of the holes may be varied or additional holes provided depending upon the actual excavation profile at any section. The exact spacing of sections may also be varied on similar considerations. It should, however, be also adjusted to suit the length of the arch shutter used in such a way that there is no hole at the joint and the normal pattern of holes is more or less uniform in the shutter length.

13.3.1.1.5 In the case of circular or horse-shoe tunnels, in addition to these holes, two holes (one on either side), located roughly at 45° on either side of the invert should be used. The location should be such that the holes are about 45 to 60 cm, above the junction of the invert and side walls or overt.

13.3.1.1.6 The mortar used for backfill grout shall normally consist of cement, sand and water mixed in the proportion of 1:1:1 by weight. It may, however, be suitably modified if conditions so warrant. The size and grading of sand should be determined for each job by actual experimentation as it would depend on the type of sand and equipment available.

13.3.1.1.7 Backfill grouting should normally be done at a pressure of 2 kg/cm^2 (0.2 MPa).

13.3.1.2 *Contact grouting*

13.3.1.2.1 The aim of contact grouting is to fully pack the space between the concrete lining and the rock surface or the space between the steel liner and concrete lining caused by shrinkage or left unfilled even after backfill grouting. This is required for fulfilling the design assumption of the rock/concrete taking part of the load along with the lining and to prevent local accumulation of water, if any, and building up local pressure.

13.3.1.2.2 Contact grouting should be done after the concrete lining has gained strength to withstand the pressure and shrinkage, if any has taken place. The usual minimum period of 25 to 28 days of waiting should be allowed.

13.3.1.2.3 The contact grouting should be limited to only the top arch (90° on either side of the crown) of tunnels. In case of vertical shafts and steel liner, contact grouting should be done along the full periphery. In case of steel-liners, the grouting should be done usually at specific points as recommended in IS 5878 (Part 6).

13.3.1.2.4 The holes at the crown shall be placed 5 to 10° from the crown, being alternately to the left and right of the crown. In addition to the crown hole, there shall be two more holes one on either side of the crown in each section. These holes will be 90° apart and will be located such that one of the two holes is at 22.5° from the crown, being alternately on the right and left of the crown. Such holes shall normally be 3 m apart.

13.3.1.2.5 In case of circular or horse-shoe tunnels, in addition to these holes, two holes (one on either side), located roughly at 45° on either side of the invert should be used. The location should be such that the holes are about 45 to 60 cm above the junction of the invert and arch.

13.3.1.2.6 The depth of holes for contact grouting shall be such that at each location, the holes extend 30 cm beyond the concrete lining into rock.

13.3.1.3 *Consolidation grouting*

13.3.1.3.1 The aim of consolidation grouting is to fill up the joints and discontinuities in the rock up to the desired depth.

13.3.1.3.2 Consolidation grouting shall always be done after the backfill grouting is completed in a length of at least 60 m ahead of the point of consolidation grouting.

13.3.1.3.3 Consolidation grouting should be usually done all round the bore, and for a uniform radial distance from the finished concrete face. The extent of grouted rock mass should be determined by the designer based on the design of the concrete lining and the extent to which cracks are assumed to extend into rock when the lining is stressed by internal pressure. Usually the depth should be between 0.75 D to D where D is the finished diameter of the tunnel, except in special reaches where it could be more.

13.3.1.3.4 The pattern of grout holes for consolidation may be a set of holes in one vertical plane, such a plane being called the grout plane. The spacing of the grout planes will depend upon the structural formation of rock and the travel of grout at the specified pressure. The exact spacing as in the case of contact grouting should also be adjusted in the field to suit the length of the shutter used for concreting. In this plane the number of holes may normally be 4 for small size tunnels and 6 for large size tunnels. The arrangement should be staggered in alternate grout planes, by about half the spacing between the holes along the periphery in the plane. In special locations the number of holes may be increased. The top three holes in grout pattern may be used for both backfill, contact and consolidation grouting.

13.3.1.3.5 Around shafts and large opening like powerhouse, the grout pattern will be similar, but the number of holes in the plane may be increased depending on the size, but the spacing should generally exceed the depth of the hole.

13.3.1.3.6 Contact grouting would not generally be necessary where consolidation grouting is to be done. However, it should be decided by actual contact grouting in jump holes after consolidation grouting.

13.3.1.3.7 Depending upon the rock formations and the grout intake, the consolidation grouting should be done in one or more stages with increasing pressures.

13.3.1.3.8 Maximum grout pressure should not normally exceed twice the design load on lining or supporting systems as the case may be.

13.4 Pressure to be Used for Grouting

13.4.1 The pressures to be used for grouting will depend on the rock characteristics, the design requirements and the rock cover. With adequate rock cover (more than 3 times the diameter of the tunnel), the other two will govern. For backfill grouting the maximum recommended pressure is 5 kg/cm^2 (0.5 MPa). For consolidation grouting a maximum pressure of 7.0 kg/cm^2 (0.7 MPa) is normally recommended but this may be increased up to 20.0 kg/cm^2 (2.0 MPa) in special cases provided that there is adequate cover and the joints in the rock are not likely to open up by this pressure. This pressure can be applied from inside the rock and not close to the concrete lining.

13.4.2 The pressure gauge should be watched constantly so that the pressure on the grout is regulated as long as grouting is in progress. Any desired increase or decrease in the grouting pressure is obtained by changing the speed of the grout pump. When the grout in the supply line becomes slugging, the grout-hole valve should be closed and the blow off valve opened so that the supply line may flush or washed.

13.5 Testing for Efficacy of Grouting

This testing may be done by drilling the holes in between the grout planes and by testing water intake in these test holes. If this is compared with the water test made before the grouting, this water will give an indication of the efficacy of grouting. Further grouting of this test hole and intake in this hole will give further indications. It is only after these tests that the engineer-in-charge may decide on increasing the number of grout planes, if required.

13.6 Capacity of Grouted Arch

$$P_{gt} = \frac{2 q_{gt} l_{gt}}{B F_{gt}} \qquad \text{...(5.0)}$$

where

q_{gt} = uniaxial compressive strength of grouted rock mass (t/m^2),

l_{gt} = thickness of grouted arch (m),

B = size of opening (m),

F_{gt} = mobilization factor of grouted arch
= $9.5\, p_{roof}^{-0.35}$, and

p_{roof} = ultimate support pressure in roof of tunnel (t/m^2).

13.7 In case of water-charged rock mass or post construction saturation of rock mass, heavy support pressure and seepage pressure may develop on the concrete lining or shotcrete lining. The extra high support pressures/seepage pressures may be taken care off by grouted arch.

14 DESIGN OF INTEGRATED SUPPORT SYSTEM

14.1 General

14.1.1 A semi-empirical approach is used to determine the capacity of support system consisting of shotcrete, reinforced rock arch, steel rib and grouted arch. In Fig. 23, the dotted line shows the effective width (l') of the reinforced rock arch. The load carrying capacity of the reinforced rock arch is dependent on the minimum uniaxial compressive strength of the reinforced rock arch.

14.1.2 Figure 23 shows that the total support pressure ($P_{roof} + U$) will be equal to the sum of capacities of shotcrete, reinforced rock arch, steel rib and grouted arch. Simple hoop action is assumed as illustrated in Fig. 23. The arch subtends an angle of 2θ at its centre which is about 90° for tunnels.

Ultimate support pressure = Total capacity of support system

$$(u + p_{roof}) = p_{sc} + p_{bolt} + p_{gt} + p_{rib} \qquad \text{...(6.0)}$$

$$= \frac{2 q_{sc} t_{sc}}{F_{sc} B} + \frac{2 q_{crm} l}{F_s B} + \frac{2 q_{gt} l_{gt}}{F_{gt} B} + \frac{p_{rib}}{S_{rib} B}$$

where

p_{roof} = support pressure and is the seepage water pressure.

NOTE — In the above equation, it is assumed that shotcrete will shear along a length of arch approximately equal to ($F_{sc}B$), where all the notations in above equation have the same meaning as described earlier.

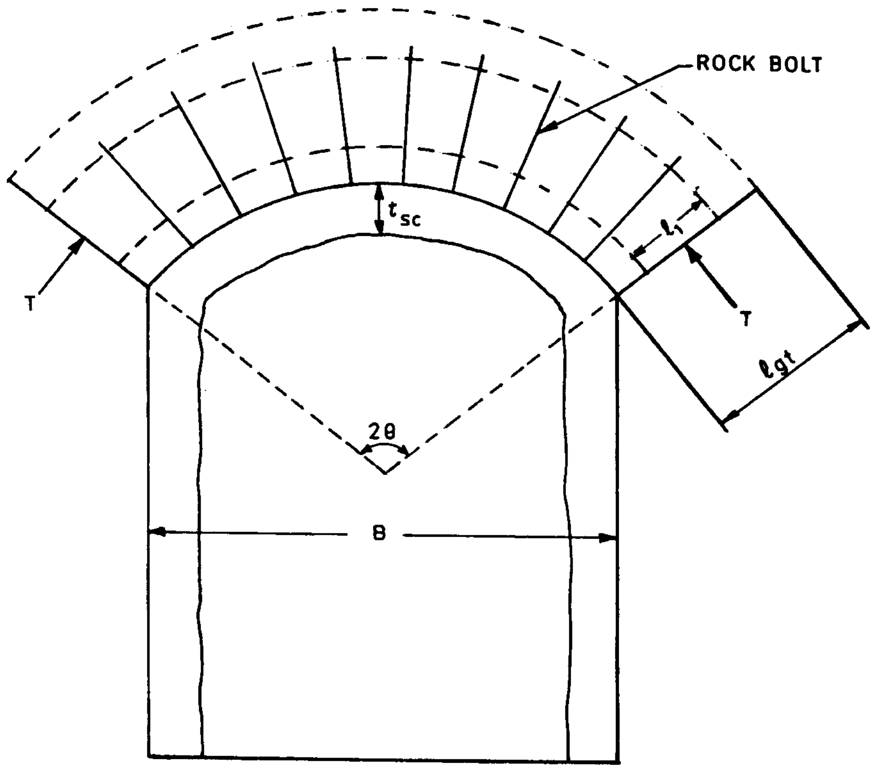


FIG. 23 CAPACITY OF REINFORCED ROCK ARCH

14.1.3 The following trends have been obtained :

a) Pretensioned bolt is more effective in good rock conditions. The efficiency of pretensioned bolt decreased slightly for poor rocks due to creep and loss of tension.

b) The full-column-grouted-untensioned anchors are more effective in poor rock conditions than in good rock conditions. The reason may be that the anchors are subjected to large radial strains in poor rock masses leading to more tension induced in anchors. Figure 24 illustrates better performance of grouted anchors.

14.2 Application of Semi-Empirical Design Approach

14.2.1 For tunnels located near faults/thrusts (with plastic gauge) in seismic areas, the ultimate support pressures may be increased by about 25 percent to account for accumulated strain in the rock mass along the fault. If the tunnel is away from the fault by $2B$, the seismic effect is negligible.

14.2.2 The support pressure, due to squeezing out of gouge from the shear zone, may be estimated by applying Terzaghi's theory of arching which indicates that the support pressure, with gauge, will increase the width of the shear zone. As such, the treatment of shear zone is essential to bear the high support pressure as shown in Fig. 5 . However, in this shear zone (say < 25 cm), the support pressure is very small and hence, there is no need for special shear zone treatment (*see* **8.1**).

14.2.3 The capacity of shotcrete and reinforced rock arch is calculated by trial and error. The design parameters are selected, so that ultimate pressure is equal to the design capacity. If support pressure is high, say more than 5 kg/cm^2, steel ribs may be used and embedded in shotcrete. The spacing of steel ribs may be estimated until Equation 4.0 is satisfied. The design philosophy is illustrated in Fig. 25. In case of water charged rock mass, experience shows that design tables give useful parameters as they neglect the seepage pressure. Equation 6.0 may then be used to find out the extent up to which rock mass should be grouted. Grouting is possible generally where thick shotcrete has been provided to take high grouting pressure.

14.2.4 In very poor rock conditions, assumptions are generally invalid. Hence, special specifications need to be followed to treat thick shear zones, rock burst conditions and highly squeezing conditions.

14.2.5 In case of water and power tunnels, seepage pressure may be assumed equal to the internal water pressure and the worst case is when the tunnel is empty and seepage water pressure acts on the shotcrete lining. If required, rock mass may be grouted to take high support and seepage pressure, alternatively, concrete lining may be designed according to the design criteria.

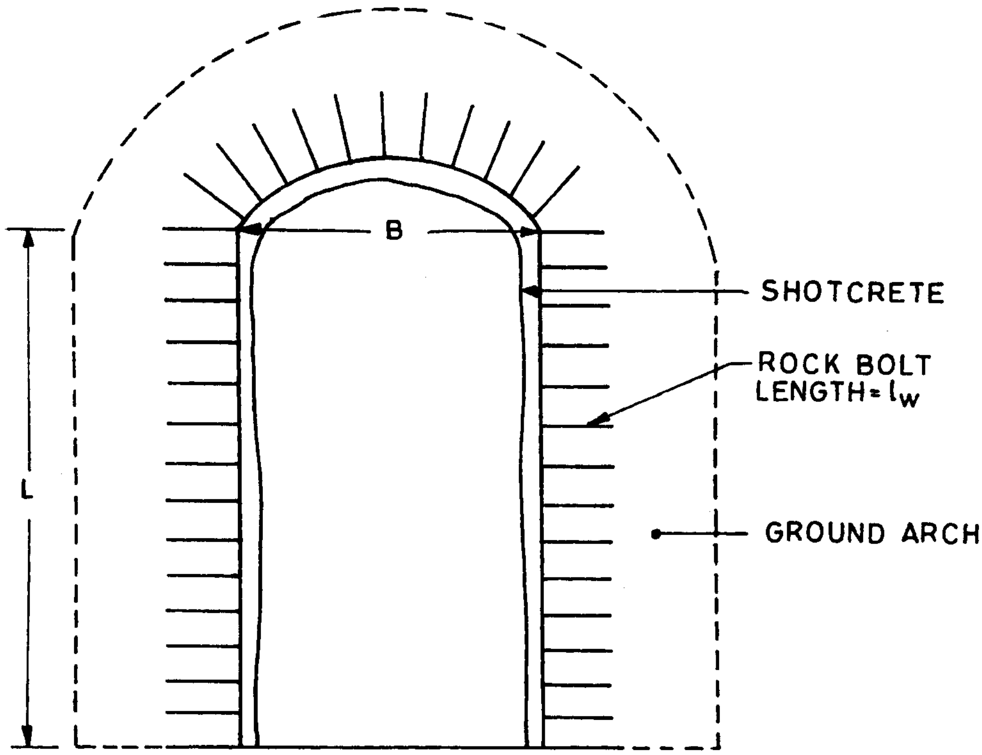


FIG. 24 DESIGN OF WALL REINFORCEMENT OF CAVERNS

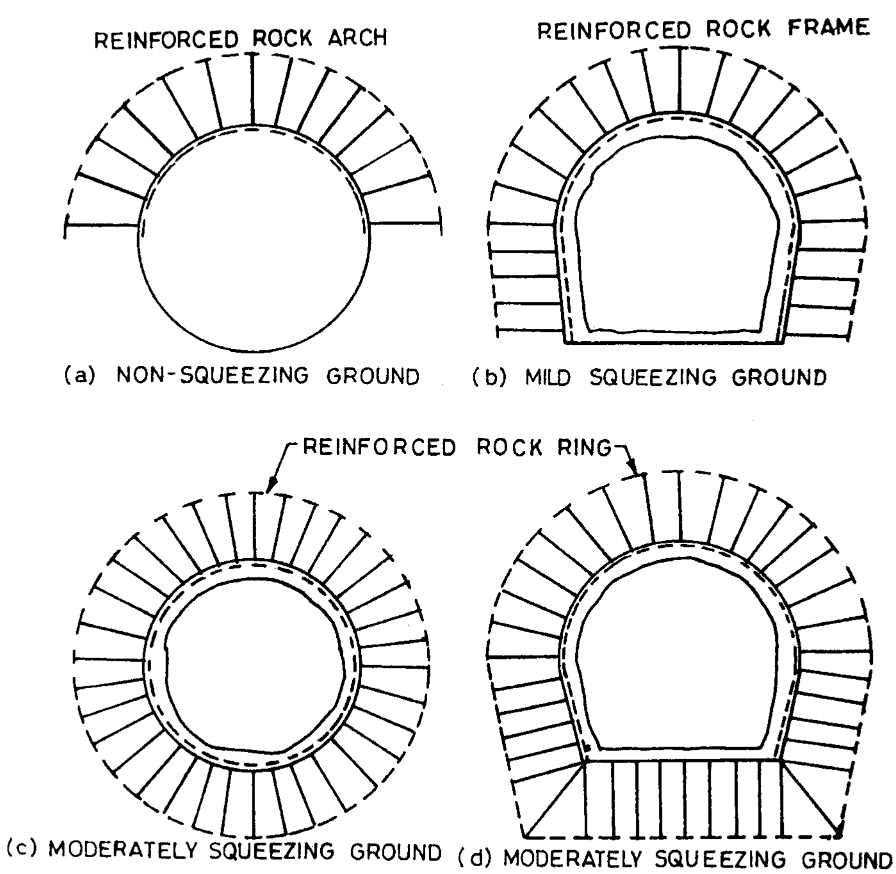


FIG. 25 DESIGN PHILOSOPHY OF ROCK REINFORCEMENT IN TUNNELING

14.3 Tunnel Through Intra Thrust Zone

In the Himalayas, the tunnel have to pass through intra thrust zone in some complex geological and tectonic situations. The faults and thrusts are subjected to slip over a long period of time due to very slow tectonic movement of Indian Plate with respect to Chinese Plate. It is therefore essential to build a segmented concrete lining so that segments of the concrete lining can slip with respect to each other with time. The design decision should be taken on the basis of instrumentation plate within the intra thrust zone.

14.4 Experience in Poor Rock Conditions

14.4.1 Fibre reinforced shotcrete is very successful in weak rock masses. The main advantage is that lesser thickness of fibre reinforced shotcrete is needed. No webled mesh is required to reinforce shotcrete. Its rebound is less due to steel fibres provided shotcrete is graded properly and sprayed properly.

14.4.2 It is recommended that the mesh should not be raised where even surface of tunnel is not available due to overbreath, provided shotcreting is used. However, thickness of shotcrete should be increased by 1 cm.

15 SPECIAL REQUIREMENTS

15.1 For treatment of shear zone, crossed rock anchors/bolts should be provided across shear zones. Further, the gouge should be cleaned to the desired extent, anchors are provided and connected to welded mesh. Final, dental shotcrete is back filled. In wide shear zone, reinforcement in shotcrete is also placed to withstand high support pressure. Anchors should be inclined according to the dip of shear zone to stop squeezing of gouge and thereby stabilize the deformations.

15.2 In case of steel ribs in large tunnels and caverns, haunches should be strengthened by additional anchors to withstand heavy thrust due to the ribs.

15.3 In case of poor rock mass, spiling bolts (inclined towards tunnels face) should be installed before blasting to increase the standup time of tunnel. Roof shotcrete is then sprayed on roof. Then spiling bolts are installed. In the final cycle, roof bolts are installed.

15.4 In case of argillaceous rocks and swelling rocks where its bond with shotcrete is poor, thickness of shotcrete may be increased by about 30 percent.

15.5 In reaches of very poor rock masses, steel ribs should be installed and embedded in shotcrete to withstand high support pressures.

15.6 In rock burst prone reaches, resin anchors and fibre reinforced shotcrete should be used to increase ductility of support system and to convert brittle mode of failure into the ductile mode of failure.

15.7 In highly squeezing ground condition ($H > 350\ Q^{1/3}$m and $J_r/J_a < 1/2$, where J_r is joint roughness number and J_a is joint alteration number [*see* IS 13365 (Part 2)] steel ribs with struts should be used when shotcrete fails repeatedly inspite of more layers. With steel ribs, excavation by force-poling is easily done by pushing steel rods into the tunnel face and welding other ends to ribs. Floor heaving can be prevented by rock bolting of floor. Some delay, but less standup time is necessary to release the strain energy of the broken zone. Smaller blast holes (1 m) will also be of help. Instrumentation of broken zone is needed. It should be ensured that the tunnel closure is arrested before it reaches the 5 percent of the width of the tunnel.

15.8 In case of unstable portal, horizontal anchors of equal length should be provided inside the cut slope, so that it acts as a reinforced rock breast wall.

15.9 In case of deep and long tunnels in complex geological conditions probe hole of 20 m length should be drilled inside the tunnel face to get accurate picture of geological conditions in advance of tunnelling. The probe hole will also dissipate seepage pressure slowly in the water charged rock mass which is likely to be punctured during tunnelling. This will also avoid flash floods soon after blasting and consequent loss of life and support system.

15.10 Concrete lining for water/pressure tunnels should be laid far away from the tunnel face where the broken zone in stabilized that is about four times the radius of broken zone. The concrete lining should be segmented within active thrust zone to allow relative movement along the faults/thrust.

15.11 Grouting of long bolts is not done satisfactorily sometimes due to the difficulty of supervision and expanding agent (Aluminium powder) is seldom added. So pull out tests should be conducted on at least 2 percent bolts to check the quality of bolts. If required extra bolting should be done to strengthen the support system.

ANNEX A

(Foreword)

COMMITTEE COMPOSITION

Rock Mechanics Sectional Committee, CED 48

Organization	*Representative(s)*
University of Roorkee, Roorkee	PROF BHAWANI SINGH ***(Chairman)***
AFCONS Infrastructure Ltd, Mumbai	SHRI A. D. LONDHE SHRI V. S. KULKARNI (*Alternate*)
AIMIL Ltd, New Delhi	SHRI M. D. NAIR SHRI B. K. SAIGAL (*Alternate*)
Central Board of Irrigation & Power, New Delhi	DIRECTOR (WR)
Central Building Research Institute, Roorkee	DR U. N. SINHA SHRI PRABHAT KUMAR (*Alternate*)
Central Ground Water Board, New Delhi	DIRECTOR AND SECRETARY
Central Mining Research Station, Roorkee	SHRI V. K. SINGH DR A. K. SONI (*Alternate*)
Central Mining Research Station, Dhanbad	DR V. K. SINHA
Central Road Research Institute, New Delhi	SHRI O. P. YADAV DR KISHORE KUMAR (*Alternate*)
Central Soil & Materials Research Station, New Delhi	DR A. K. DHAWAN
Central Water and Power Research Station, Pune	DIRECTOR SHRI B. K. RAME GOWDE (*Alternate*)
Engineer-in-Chief's Branch, New Delhi	SHRI D. K. JAIN SHRI V. S. ARORA (*Alternate*)
Geological Survey of India, Lucknow	SHRI R. P. S. CHAUHAN
Gujarat Engineering Research Institute, Vadodara	DR V. S. BRAHMABHATT SHRI M. T. SONI (*Alternate*)
Himachal Pradesh State Electricity Board, Shimla	DR R. L. CHAUHAN
Hindustan Construction Co Ltd, Mumbai	ENGINEERING MANAGER
Indian Geotechnical Society, New Delhi	DR K. S. RAO
Indian Institute of Technology, New Delhi	DR G.V. RAO
Irrigation Department, Government of Maharashtra, Nasik	DR R. P. KULKARNI
Irrigation Department, Government of Gujarat, Gandhinagar	SHRI K. N. SHAH SHRI A. N. JOSHI (*Alternate*)
Irrigation Department, Government of Haryana, Chandigarh	CHIEF ENGINEER (R&D) DIRECTOR (ENGG.) (*Alternate*)
Irrigation Research Institute, Government of Uttar Pradesh, Roorkee	ASSISTANT RESEARCH OFFICER
Irrigation and Power Department, Chandigarh	SHRI KARMVIR
Karnataka Engineering Research Station, Karnataka	SHRI S. VENKATARAMANA SHRI JAGANATHA RAO (*Alternate*)
Naptha Jakri Power Corporation, Shimla	SHRI RANJODH SINGH
National Geophysical Research Institute, Hyderabad	SCIENTIST-IN-CHARGE
National Thermal Power Corporation Ltd, Noida	SHRI D. N. NARESH DR R. R. MAURYA (*Alternate*)
University of Roorkee, Roorkee	DR P. K. JAIN DR M. N. VILADKAR (*Alternate*)

(Continued on page 26)

(*Continued from page* 25)

Organization	*Representative(s)*
In Personal Capacity (*ATES, AIMIL Ltd,* Delhi) *Mathura Road, New Delhi 110 044)*	DR V. M. SHARMA
BIS Directorate General	SHRI S. K. JAIN, Director & Head (Civ Engg) [Representing Director General (*Ex-officio*)]

Member-Secretary
SHRI D. K. AGRAWAL
Joint Director (Civ Engg), BIS

Underground Opening and Field Monitoring Subcommittee, CED 48 : 3

University of Roorkee, Roorkee 247 667	DR H. S. BADRINATH (***Convener***)
AIMIL Ltd, New Delhi	SHRI M. D. NAIR SHRI B. K. SAIGAL (*Alternate*)
Advanced Technology & Engineering Services, New Delhi	SHRI B. DASGUPTA
Central Building Research Institute, Roorkee	SHRI A. GHOSH SHRI Y. PANDEY (*Alternate*)
Central Mining Research Institute, Roorkee	DR R. K. GOEL
Central Mining Planning and Design Institute, Ranchi	SHRI S. CHAKRABARTY DR M. M. SOM (*Alternate*)
Central Soil & Materials Research Station, New Delhi	DR A. K. DHAWAN SHRI RAJEEV KUMAR (*Alternate*)
Central Water Commission, New Delhi	SHRI K. R. SUBRAMANIAN DIRECTOR (DIV II) (*Alternate*)
Geological Survey of India, Kolkata	SHRI A. BHATTACHARYYA SHRI S. K. MUKHOPADHYAY (*Alternate*)
Gujarat Engineering Research Institute, Vadodara	SHRI U. D. DATIR
Indian Institute of Technology, Mumbai	DR G. VENKATACHALAM
Indian Institute of Technology, New Delhi	DR K. G. SHARMA
Indian Institute of Technology, Kanpur	DR SURESH CHANDRA
Jaiprakash Associates Pvt Ltd, New Delhi	SHRI D. G. KADKADE SHRI R. K. JAIN (*Alternate*)
Konkan Railways, New Mumbai	SHRI NARAYANAN
Kvaerner Cementation India Ltd, Kolkata	SHRI P. S. SENGUPTA SHRI MANISH KUMAR (*Alternate*)
M.S. University, Vadodara	DR A. V. SHROFF
Maharashtra Research and Physics Division, Nasik	RESEARCH OFFICER
National Institute of Rock Rock Mechanics, Kolar	SHRI N. M. RAJU SHRI B. SHRINGARPUTTA (*Alternate*)
University of Roorkee, Roorkee	DR M. N. VILADKAR DR SUBHASH MITRA (*Alternate*)
Visvesvaraya Regional College of Engineering, Nagpur	DR A. G. PAITHANKAR

Bureau of Indian Standards

BIS is a statutory institution established under the *Bureau of Indian Standards Act*, 1986 to promote harmonious development of the activities of standardization, marking and quality certification of goods and attending to connected matters in the country.

Copyright

BIS has the copyright of all its publications. No part of these publications may be reproduced in any form without the prior permission in writing of BIS. This does not preclude the free use, in the course of implementing the standard, of necessary details, such as symbols and sizes, type or grade designations. Enquiries relating to copyright be addressed to the Director (Publications), BIS.

Review of Indian Standards

Amendments are issued to standards as the need arises on the basis of comments. Standards are also reviewed periodically; a standard along with amendments is reaffirmed when such review indicates that no changes are needed; if the review indicates that changes are needed, it is taken up for revision. Users of Indian Standards should ascertain that they are in possession of the latest amendments or edition by referring to the latest issue of 'BIS Catalogue' and 'Standards: Monthly Additions'.

This Indian Standard has been developed from Doc : No. CED 48 (5597).

Amendments Issued Since Publication

Amend No.	Date of Issue	Text Affected

BUREAU OF INDIAN STANDARDS

Headquarters :

Manak Bhavan, 9 Bahadur Shah Zafar Marg, New Delhi 110 002
Telephones : 323 01 31, 323 33 75, 323 94 02

Telegrams : Manaksanstha
(Common to all offices)

Regional Offices :		Telephone
Central	: Manak Bhavan, 9 Bahadur Shah Zafar Marg NEW DELHI 110 002	323 76 17 323 38 41
Eastern	: 1/14 C.I.T. Scheme VII M, V. I. P. Road, Kankurgachi KOLKATA 700 054	337 84 99, 337 85 61 337 86 26, 337 91 20
Northern	: SCO 335-336, Sector 34-A, CHANDIGARH 160 022	60 38 43 60 20 25
Southern	: C.I.T. Campus, IV Cross Road, CHENNAI 600 113	254 12 16, 254 14 42 254 25 19, 254 13 15
Western	: Manakalaya, E9 MIDC, Marol, Andheri (East) MUMBAI 400 093	832 92 95, 832 78 58 832 78 91, 832 78 92

Branches : AHMEDABAD. BANGALORE. BHOPAL. BHUBANESHWAR. COIMBATORE. FARIDABAD. GHAZIABAD. GUWAHATI. HYDERABAD. JAIPUR. KANPUR. LUCKNOW. NAGPUR. NALAGARH. PATNA. PUNE. RAJKOT. THIRUVANANTHAPURAM.

Printed at Prabhat Offset Press, New Delhi-2